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THE RECORD OF THE FIRE-PROOF BUILDING IN THE BALTIMORE CONFLAGRATION.

The great conflagration which swept the city of Baltimore on February 8 and 9, and laid in waste its entire wholesale business and banking district, is estimated to have destroyed property valued at about $125,000,000. No array of figures can express the indirect financial losses. These are represented by hundreds of businesses destroyed, thousands of well-paying positions abolished, and millions of dollars worth of trade abruptly suspended. To meet these losses there is only the insurance, and this amounts, with all claims proved and paid, to about $65,000,000. Measured by these figures of financial loss, the Baltimore fire ranks, probably, as the second greatest conflagration ever recorded. In loss of life and in the amount of destitution involved, however, it was, happily, less disastrous than many fires of insignificant proportions. One man killed and barely a score injured to the astonishingly small record of human casualties.

It is in its financial losses, direct and indirect, that the Baltimore fire looms above all previous conflagrations, except the great Chicago fire of 1871, and here it approaches the magnitude of a national calamity.

To many persons the Baltimore fire seems to have put the question whether the American city of to-day can be so builded as to be safe against such fires as those at Chicago in 1871, and at Boston in 1872, and to have answered it in the negative. The 150 acres of black and smoking ruins which were once the most substantially built portion of the sixth city of the United States, permits at first sight no other conclusion. Already, on this showing alone, the public press has widely condemned the modern type of fireproof building, and some even whose words are weighted with expert authority in the public mind, have called for a return to "brick and mortar" as the only salvation of the building owner when conflagration besets his property.

A careful study of the burned district throws much light on this subject; and it will be wise for engineers to be in no haste to accept these statements of ruin until they have the results of such an investigation from which to reason.

There are many lessons to be drawn from the Baltimore fire, and some of them are not soothing to the vanity of the engineer as a builder; but others are a magnificent testimonial to the perfection of his work. It is for these reasons that we give in this issue a story of the fire and of the destruction wrought by it, which is based upon a careful study by editorial representatives, who witnessed the progress of the fire on Feb. 9 and examined its ruins in detail afterwards, and by independent experts in fireproof building construction.

A TYPICAL SCENE IN THE BURNED DISTRICT OF BALTIMORE; CHARLES ST., LOOKING SOUTH.

ORIGIN AND PROGRESS OF THE FIRE.

The Baltimore fire originated in the dry goods store of Hurst & Co., which was located at Hopkins Place and German St., the point indicated by the white cross on the accompanying sketch map (Fig. 1). To the north and west of this point lay the most beautiful residential district of the city, and to the south and east were located the wholesale business and financial district and all the large office and municipal buildings. According to the automatic fire alarm, the fire started at 11.48 a. m. on Sunday morning. The response to the alarm was prompt, but meanwhile the flames had gained considerable headway on the lower floors.

The building has been described in some of the published accounts as of fireproof construction. This is not true. It was a six-story building, with brick walls and floors of I-beams carrying wooden joists and flooring on cast-iron columns, and every story was packed with inflammable dry goods. In ten or fifteen minutes after the alarm this combustible fuel was a seething mass of flames on every floor and the floors themselves were burning. No hope of saving the building existed even at this early stage, and the firemen's endeavor was to prevent the further spread of the fire.

As the floors fell one after another, the flames rose high above the roof and burning embers began to be carried by the draft and wind which rained them in showers on the adjoining buildings. A series of explosions, caused either by bursting gasoline tanks or steam boilers, aided the work of the wind by hurling the burning debris into its grasp. In an hour a whole city block was in flames and the fire was spreading to the adjoining blocks south and east. From this time on the story of the fire is a simple one.

Baltimore, like most American cities, is structurally a mingling of the old and the new. Wooden and brick veneered buildings alternate with fireproof office buildings of structural steel and terra cotta with here and there a low bank building of granite and brick, with almost the solidity of an Egyptian temple.

As in most American cities, these fireproof buildings are surrounded by inflammable buildings, dwelling houses converted into stores, brick and stone shells with wooden floors, partitions

and interior trim. Into this heterogeneous mixture of buildings the fire ate its way under the impluse of the 30-mile gale from the northwest Six hours after the Hurst Building had begun to burn the flames were crossing Charles St., and everything between that point and the origin of the fire was a blazing ruin. In six hours, therefore, the fire had devastated eleven city blocks, and its steady progress had not once been checked.

The fighting line of fire companies, which had up to this time been squarely drawn began to waver and soon the attacking flames had overrun the defense. The fire, which hitherto advanced slowly, now began to progress in leaps and bounds. To meet this movement resort was made to razing the buildings in the path of the advancing flames. This work was of no avail, the fire simply leaped the gaps as it had leaped the streets. Building after building and block after block caught fire and burned and lent torch to the next building and block. All through the night and all day Monday until 5 o'clock in the afternoon the destruction continued. Then the struggling firemen succeeded in quenching the advance guards of flame which had not already been checked by the harbor basin and the channel of Jones Falls.

The fire had lasted 30 hours and in that time had burned over an area of 150 acres, every acre of which was occupied by buildings and merchandise. Altogether some 2,000 buildings were burned.

The devastated district is shown by the accompanying map. As will be seen the course of the fire was almost directly east and its path was about four blocks in width. During its early stages the good work of the firemen kept the barriers drawn at Pratt St. on the south and at Fayette St. on the north; afterwards the Harbor Basin came to the aid of those who were fighting along the southern line of battle and on the north the massive granite court house, post office and city hall furnished an almost equally impregnable line of defense. To the east the fire swept unchecked until the little stream, half canal and half creek, gave a line of natural defense in that direction and Jones Falls marked the eastern boundary line of destruction.

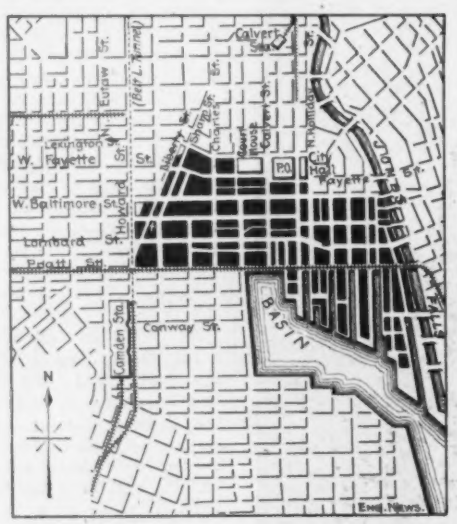


Fig. 1. Sketch Map Showing Burned District, Baltimore, Md.
(White Cross Indicates Origin of Fire.)

Though the story of the fire has been told in detail in every newspaper, we have been somewhat circumstantial in describing its progress for two reasons. The first of these is to show how irresistible a modern city conflagration may become when once it has gained headway; the efforts of the best of fire fighting devices and personnel are puny in comparison with the force against which they offer protection. The second reason is to demonstrate the exceeding combustibility of the average modern city of the first class. This feature will be further emphasized as the story of the complete destruction wrought by the flames is told. These are two lessons taught by the Baltimore fire which cannot be too thoroughly mastered by those having the administration of municipal affairs.

EXTENT AND CHARACTER OF DESTRUCTION.

The accounts which have gone abroad have stated that the destruction wrought by the fire was complete, and they are not far from the truth for the reason that 99% of the buildings in the burned district were of the kind whose destruction was a foregone conclusion if ever a conflagration encountered them. There were a few buildings, however, which were not destroyed, because they were built to withstand fire, and, so far as these buildings are concerned, the destruction was not complete. These are facts which must be grasped by everyone who wishes to have the true record of the fireproof building in the Baltimore fire.

Unfortunately the daily newspapers have failed to grasp these facts, and the story has gone abroad that fireproof building construction is a failure when tried by fire. To one who watched these buildings endure the flames at Baltimore, or who sees them now looming up, with unbroken walls from pavement to cornice, a solitary figure in a scene of chaotic destruction and desolation like that depicted by our first page picture, they must excite admiration and pride in the structural soundness that enabled them to stand where all else was leveled to the ground. Whatever the structural integrity of these buildings may now be, they at least merit this praise that they still stand upright amidst the heaps of brick and mortar which mark the sites of former neighboring structures. It will be shown further on that they have other claims to commendation, but, first, attention will be called to the behavior of some of the older types of structures.

NON-FIREPROOF BUILDINGS.—In the classification adopted here non-fireproof buildings comprehends all classes of buildings in which wood was used structurally or in which the ironwork was not protected by fire-resisting material or by fireproofing. At Baltimore these buildings ranged from combustible timber structures to buildings with brick or stone exterior walls and floors of I-beams carrying timber joists and flooring on cast-iron columns. The engineer and architect will readily recognize the class of city buildings which comes within these limits. Such buildings form the great bulk of every city; in many cases they represent enormous individual expenditures of money. They are not fireproof buildings in the modern meaning of that term, scarcely more than the outer walls is even incombustible.

In the Baltimore fire the non-fireproof buildings suffered total destruction almost without exception. Of the very best of them only fragments of the outer walls remained standing. The ruins of the Law Building, shown by Fig. 2, and of the American Building, shown by Fig. 3, are typical examples of the destruction suffered by the most costly of these structures. Those of less sturdy construction are represented by the heaps of brick and jagged spires of wall shown by the first page picture and the view along Baltimore St.

As has already been noted, over 99% of the 2,000 odd buildings in the burned district consisted

VIEW SHOWING BRICK STORES ON BALTIMORE ST., LOOKING EAST.

of buildings of this class. Their destruction was complete and is accountable for the vision of ruin which the burnt district presents for block after block.

MONUMENTAL BUILDINGS.—The buildings of truly monumental character, which came within the path of the fire were few in number. The best examples were the Court House, the Post Office and the City Hall. These were all structures of moderate height, with thick exterior walls of granite and floors of brick arches or terra cotta carried by heavy interior walls of brick, or by brick piers and cast-iron columns. The window area was moderate and each building was bounded on all four sides by streets. The Court House was exposed on two sides to the flames of burning building across the adjoining streets; the Post Office and the City Hall were each exposed to fire on one side only. The fire did not gain entrance to any of these buildings against the streams of the building hose. These streams were directed against the window casings as fast as fire caught them, and the defense had to be continued as long as the opposite buildings continued to burn. The full measure of the damage to these buildings was the destruction of most of the windows on the exposed sides and the spalling and cracking of the granite walls. The greatest damage was suffered by the Court House which bore the fiercest attack.

In addition to resisting serious destruction, the three buildings under consideration can justly claim the credit for much of the success with which the spread of the fire northward was prevented. For three blocks they formed veritable Gibraltars against which the fire flung its attacks in vain. So great was the advantage thus gained that the firemen successfully maintained the northern line of defense set by them as the flames progressed the three remaining blocks eastward to Jones Falls.

The behavior of these buildings under the attack of fire should not be dismissed without mention of the detail failure of the granite facades by spalling. This action was particularly noticeable on the west facade of the Court House. Here the projecting cornice, the window seats and lintels and in fact all exposed corners and thin edges of stonework were broken and splintered beyond repair short of reconstruction. The same be-

havior was recorded by stone walls and facing in every building examined in the burnt district. At the new Custom House, under construction a block or two south and east of the City Hall, the partly reared granite walls were scarred and cracked like an ancient ruin. Chips from the size of the thumb nail to lenticular pieces a foot or

FIG. 2. A BUILDING WITH INDEPENDENT WALLS AND I-BEAM AND TIMBER FLOORS.

FIG. 3. REAR OF CONTINENTAL TRUST BUILDING, WITH BALTIMORE "AMERICAN" OFFICE IN FOREGROUND.

more in lateral dimensions and two or three inches thick at the center completely covered the ground at the base of the walls. The moldings and ornamental carvings of stone which embellished the tall office building entrances were in all cases similarly broken and scarred. The view in Fig. 4 of a granite pillar supporting a warehouse floor is a fair indication of the damage which may result from spalling. This pillar was some distance from the source of flame but was subjected to intense heat and as will be seen was seriously damaged. It is perfectly safe to say that in all the burned office buildings at Baltimore, the cost of restoring to perfect condition the first three or four stories with stone facing will be nearly as great as the cost of all the remaining exterior wall restoration.

FIREPROOF BUILDINGS.—The fireproof buildings in the burned district may be divided into two classes, those of the construction prevailing from 15 to 20 years ago and those of the more modern types of the last five or ten years. In the former class it is proper to include the Chamber of Commerce and the Baltimore & Ohio R. R. Office Building.

The Chamber of Commerce was built some 25 years ago. It had heavy exterior walls of brick with stone trim that carried a truss roof spanning the "pit" or trading room which occupied the whole top of the building. The floors consisted of tile arches carried between I-beam girders supported by heavy partition walls and cast-iron columns. The effects of the fire on the exterior walls was confined to the spalling of the stone trim, except in the case of the rear party wall which had been partly torn out by the collapse of the adjacent building. The roof was entirely gone. The heavy brick partitions were intact, but the cast-iron columns were out of position, and in two instances on the ground floor round columns of cast iron about 15 ins. in diameter had bent at least 2 ins. from a straight line between their ends. It may be noted here that one of the most surprising effects of the fire was the noticeable extent to which cast iron had suffered permanent bends. This effect was most noticeable in the cast-iron mullions of the tall office building windows. These in many cases were bent several inches from a straight line and in some cases there bends were veritable "buckles" of the metal.

The Baltimore & Ohio office building was of unusually substantial construction. The outer walls were of brick with heavy stone trim, and the interior was of very similar construction to the Chamber of Commerce. Practically the only dam-

Fig. 4. An Example of Stone Spalled by Heat; a Granite Warehouse Pillar.

age done to the outer walls was the spalling of the stone trim and this was badly pitted and broken. The roof remained in excellent condition, and the partitions and floors generally were intact. A view of this building is shown by Fig. 5.

Next in point of age to the Baltimore & Ohio building was the Equitable building, constructed in 1892. This is a steel skeleton fireproof structure of ten stories, and it was the worst damaged of any of the tall buildings. From top to bottom every floor panel for about half the area of the building had collapsed, and the basement was littered with safes precipitated from the offices above. The floors were of terra cotta, end construction arches. The column protection showed only detail injuries except in a few cases where it had been broken away by the falling floors. The partitions, of some sort of plaster composition on metal mesh, were totally destroyed. Despite the somewhat sad wreck of its interior this building shows little exterior damage beyond the spalling of its stone facing and trim. Many of the floor beams will have to be replaced, and all the partitions and broken floor arches. A goodly proportion of the remainder of the fireproofing can be repaired. A view of this building is shown by Fig. 5.

Adjacent to the Equitable Building is the Calvert Building, constructed about three years ago. This is a steel skeleton fireproof structure with terra cotta fireproofing. It was of unusually heavy construction and came through the fire structuraly intact with the exception of one badly buckled column on the eighth floor, and occasional detail failures in the fireproofing. The exterior walls showed little damage.

A somewhat older structure of similar construction is shown by the view Fig. 6. This is the new building of the Baltimore "Herald," and it came through the fire in good condition. None of the structural steel work suffered visible injury and practically all of the fireproofing terra cotta was intact. A very moderate sum will put the skeleton structure and fireproofing in good condition.

The Union Trust Building located at Fayette and Charles Sts., and the Maryland Trust Building at German and Calvert Sts., are about the same age and size and of practically similar construction. Both are about four years old and are eleven stories high, with stone facades for two or three stories. Above the second story the exterior of the Maryland Trust Building is ornamental terra cotta. This, owing to its highly ornamental form, broke and chipped in many places, but was in much better condition and far superior in appearance to the stone facing below, all of which

had spalled and cracked seriously. In the Union Trust Building the stone facing had suffered in the same way, but the brick walls above were in excellent condition. Both of these buildings were of steel and terra cotta construction. The steel work showed no evidence of general injury and except for detail failures the protecting terra cotta was in good condition.

The largest of the Baltimore tall buildings was the Continental Trust Building, 16 stories high, located at Baltimore and Calvert Sts. This building was completed about a year and a half ago and was claimed by its owners and builders to be of the most recent and approved type of cage construction in all details. A rear view of this building is shown in both Figs. 5 and 6. This is the most seriously damaged facade and to the layman presents a rather terrifying spectacle. In reality, however, the damage done is quite superficial. The window mullions of cast iron and some of the sills and curtain beams were warped and twisted and the brick veneer in places and much of the plastering had peeled off. These were not structural parts; indeed, the structural framework is nowhere exposed except where the lower beam flanges are bared in places. The column protection is everywhere intact and the partitions are generally in place. The floors in a number of cases show breaks through the lower flanges of the tile, but few of them have failed completely even for a single panel. Generally speaking the building as a structure is intact as to skeleton, and only a little damaged as to fireproofing.

In all of the fireproof buildings which have been named the entire contents were destroyed; every window had been taken out; all the ornamental and marble work, the wooden floor strips, glass and trim and all the fixtures had been destroyed; practically every combustible particle of the structure was in ashes. The heat in places had been sufficient to soften cast iron until it bent like putty and to fuse typewriting machines and ornamental fixtures into solid masses. In one case only, were the windows protected by fireproof shutters. The rear wall of the Maryland Trust Co. had underwriters' shutters of wood sheeted with steel plate at every window. These had been closed before the building took fire, but it is impossible to ascertain what protection they offered since the building was ignited from the front and side windows. They were, however, subjected to fire on both sides, and under this attack most of them had burst open, all were badly warped and twisted, and some of them hung in ribbons.

Strictly speaking the only buildings of steel cage fireproof construction in the Baltimore fire were the Continental Trust, the Maryland Trust, the Equitable, the Union Trust, the Calvert and the Herald. In all of these the steel framework is intact and plumb and with a new member in one or two places will be as good as new; in all of them, the exterior walls are structurally sound, although exhibiting surface blemishes due to spalling in all but one, 90% of the floor arches are intact, and of the remaining arches not more than a quarter will have to be wholly rebuilt; in all, every window is gone, and every item of finish and contents has been destroyed; in all, ignition resulted from the flames of adjacent non-fireproof buildings breaking through the unprotected windows. This is the record of the fireproof buildings in the Baltimore fire. In a succeeding issue we shall present a further discussion of the wreck caused by the Baltimore fire and of the lessons that may be drawn from it.

FIG. 5. A GROUP OF BALTIMORE'S FIREPROOF BUILDINGS AFTER THE CONFLAGRATION.

THE PROPOSED TUNNEL FOR WATER DEVELOPMENT FOR SANTA BARBARA, CAL.*

The city of Santa Barbara, in Southern California, is situated on a narrow coastal plain, backed by a high range of mountains. It is a locality blessed with great beauty, being sometimes called the American Riviera, but with naturally a deficient water supply. The streams coming off the coastal face of the Santa Ynez Range are small in drainage area and flashy in regimen.

This Santa Ynez range of mountains consists of shale and sandstone, the strike of the formation being parallel to the coast and the dip practically vertical. The rocks are fissured, and there is stored therein a material quantity of water. The city of Santa Barbara has run a tunnel line 5,000 ft. in length into this range of mountains. It was found that, during the time when this tunnel was being actively extended, a sufficient quantity of water was "developed" to supply the greater portion of the city; in other words, that, as the tunnel progressed, one fissure after another was opened, which, in draining off, maintained a substantial flow from the tunnel. When work was suspended, this supply materially decreased. In order to regulate and control this, the interesting experiment was adopted of building a masonry bulkhead near the outer end of the tunnel, and inserting therein a pipe with a gate so that the tunnel could be tightly closed. This was not placed in time to get the full benefit of the winter rains, but the opening was partly closed before the end of the so-called wet season. The rain waters accumulating in the fissures in the rocks were again stored therein. A pressure gage placed in this bulkhead indicated that the tunnel not only filled, but that a "head" of 100 ft. was accumulated in the fissures, notwithstanding the fact that the flow was not entirely cut off, but only reduced from about 500,000 to 250,000 gallons per day. During the rainy season, the local streams furnished an adequate supply for the city. As the rainy season passed, the gate in the tunnel dam was slowly opened and this stored water was used during the past season to successfully carry the city through the dry portion of the year, the output being about 250,000 gallons daily for 200 days, after which the gate was entirely opened and a decreased supply continued. The town of Santa Barbara, however, is growing so rapidly that an additional water supply is urgently needed.

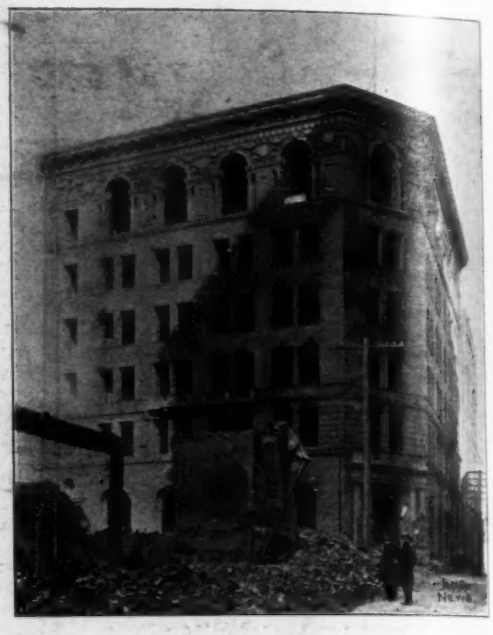


Fig. 6. The Fireproof "Herald" Building, Constructed in 1898.

The U. S. Geological Survey has made a general study of the water supply of Southern California, and of the Santa Ynez River in particular. The stream flows westerly on the northern side of the Santa Ynez range of mountains, entering into the Pacific Ocean some 80 miles northwesterly from the town of Santa Barbara. These investigations indicated that this stream discharged a large amount of winter flood water, which passed on unused to the sea, although, during the summer time, the river went entirely dry. At a point north of Santa Barbara, the Santa Ynez River basin contains 220 square miles. Reservoir sites of unusual value occur therein, and the impounding of the winter water would furnish an adequate supply, not only for the domestic use of the city of Santa Barbara, but also for the irrigation of a large area of suburban land, provided the water could be delivered from the reservoir sites to the city. Benefiting by the investigations of the U. S. Geological Survey, surveys have since been made by the city over this range of mountains, showing the possibility of diverting the flood waters of the Santa Ynez River from these reservoirs to Santa Barbara by means of a tunnel 19,500 ft. in length. There will be no shafts, and the tunnel will be run from two headings. Estimates have been prepared on the cost of the entire work and the city has voted the bonds to build the tunnel, which is the first stage in the general water development. The construction work will probably begin within the next 60 days and the city will receive bids for the tunnel until Feb. 23. It will be 7 ft. high, 5 ft. wide on the base, and 3 ft. 6 ins. wide at the top. It is estimated that it will take approximately five years to construct, and that, during the time it is being run, sufficient water will be "developed" to sustain the city, so that when the tunnel is completed, the flood waters of the Santa Ynez River can be turned therein. The construction

*For this statement we are indebted to Mr. J. B. Lippincott, M. Am. Soc. C. E., Resident Hydrographer U. S. Geological Survey, Byrne Bldg., Los Angeles, Cal.

of reservoirs on the Santa Ynez River will follow. It is to be hoped this work will solve the water problem for this beautiful coastal district.

PROGRESS OF ENGINEERING WORK FOR THE ST. LOUIS EXHIBITION.

A detailed account of the varied and extensive construction work for the Louisiana Purchase Exhibition at St. Louis was given in our issue of April 23, 1903, and we give below a summary of the progress of this work up to Feb. 1, 1904, from a statement kindly prepared for us by Mr. Richard H. Phillips, Chief Engineer of the Department of Civil Engineering.

SEWERS.—The storm water drains, including the main channel of the River Des Peres, have all been finished at a total cost of $236,706. The sanitary sewers, varying in size from 6-in. to 30-in. vitrified pipe and 4-ft. brick sewer, have been completed at a total cost of $93,825. There are a few concession restaurants yet to be located, which will require some runs of vitrified pipe, but the amount to be done is small. The sewage pumping station has been in commission for a number of months. The full installation of four 12-in. centrifugal pumps with four 75-HP. motors has not been entirely completed, however, but it is expected that this will be completed and in full operation by March 15. The pumping station, including pumps, motors and the companion pavilion, cost a total of $30,285.

WATER SUPPLY.—The high-pressure system of water pipes, costing $258,382, is 96% done. This work includes all pipe in streets, pipe in buildings, roof pipe, sprinkler system in high towers, and sprinkler system under the elevated portion of the Fine Arts Building. Four fire engine houses have been built and fully equipped, two of them being for double companies.

PAVING.—For asphalt paving, the amount expended has been $83,000. There is a small amount of this work yet to be done on account of the location of special pedestals for statuary, balustrades of bridges, etc. The "Pike," along which most of the concessions have been located, has been paved with brick at a total cost of $46,876. This street is 90-ft. wide and has a 3-ft. cement gutter down the center. There is a slight drop from the building line of the concessions to the center of the street. Ornamental lamp-posts are placed 75 ft. apart along the "Pike," with a high-pressure hydrant near each lamp-post.

The roads built of macadam have been completed, though, of course, a great deal of street cleaning will be necessary and considerable renewals and finishing work will have to be done after all the construction work is completed. For this purpose, shipments of macadam of 1½-in. and 2½-in. size are delivered regularly. The exposition company has stored and is receiving sufficient burnt-clay ballast for surfacing the macadam roads which are to be used as walks during the exposition period. At present these roads are surfaced with macadam only, and will not be given their final dressing until just before the opening. The total amount expended on macadam roads is about $150,000; telford roads, $17,923, and clay ballast, $12,000.

Grading to the amount of 2,161,089 cu. yds. has been done, the total cost being about $392,247.

LAGOONS.—Eight lagoon bridges have been completed at a total cost of $75,000. Lagoon revetment and lagoons calculated to hold 20,000,000 gallons of water, and having an area of 750,000 sq. ft., have been completed at an expense of $85,000. The bottom of the lagoons has been covered with macadam at an expense of $20,000. The timber work, cement steps, cascade basins, fountains, etc., have been completed at an expense of $110,000. These figures on the lagoons and cascades, however, do not include the ornamental staff work, metal urns and plaster finish, which have been completed at a cost of $152,000. This last figure, however, includes four small bridges across the south lagoon.

RAILWAYS.—The World's Fair terminals, with 19.4 miles of track, including classification yards, etc., have cost $338,225. There are a few of these tracks yet to be placed, but they are built as needed. The Intramural Railway, not including stations, has been completed at a cost of $103,883. Part of the tracks now being used by the World's Fair terminals for switching purposes will be put into use for the Intramural tracks. This will require a few new connections and crossovers, material for which is on hand and which will be put in promptly when it is necessary.

BUILDINGS.—The main buildings are all practically completed and the installation of exhibits has begun in earnest.

COMBINATION DINING, PARLOR AND DAY CARS.

A matter which comes up periodically for discussion in railway circles is the provision of parlor and dining-car accommodation on fast trains over what may be termed secondary routes, where the patronage of this class of service is limited, but does exist to some such extent as to warrant its consideration. To haul a heavy parlor car and a dining car on a route where only a few people would use them is a very expensive proceeding, and yet there may be a certain demand for accommodations superior to those of the ordinary day car and the lunch rooms at country stations.

This problem is being solved on some roads by the use of combination cars, and these are usually operated by the railway companies and not by private car companies. On the Chicago and Cincinnati route formed by the Chicago, Indianapolis & Louisville Ry. and the Cincinnati, Hamilton & Dayton Ry., two types of combination cars are used. One of these is a combination day and parlor car, with toilet rooms for both sections, and a smoking room for the use of passengers in the parlor section. The other is a combination dining and parlor car.

A view of the combination day and parlor car is shown in Fig. 1, while Fig. 2 is a floor plan. The car is 70 ft. long over the body and sills, and 77 ft. 6 ins. over the platforms, which are enclosed by wide vestibules of the modern type, as shown. At one end is the parlor section, 24 ft. 3 ins. long, with 12 revolving chairs and a sofa. At the other end is the ordinary day section, 23 ft. 5 ins. long, with 16 high-backed seats. The toilet rooms for this section are at the extreme end of the car. At the middle of the car are the smoking room and toilet rooms for the parlor section, a corridor affording communication between the two sections.

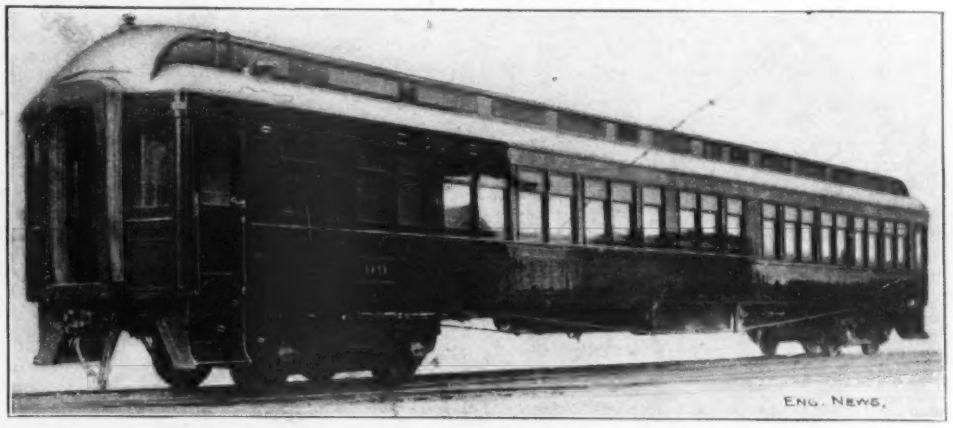


FIG. 1. COMBINATION DAY AND PARLOR CAR; CINCINNATI, HAMILTON & DAYTON RY.
Barney & Smith Car Co., Dayton, O., Builders.

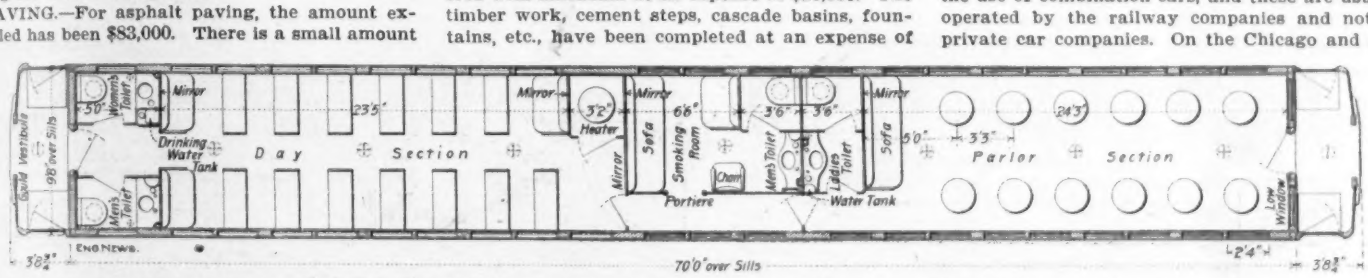


FIG. 2. PLAN OF COMBINATION DAY AND PARLOR CAR; CINCINNATI, HAMILTON & DAYTON RY.

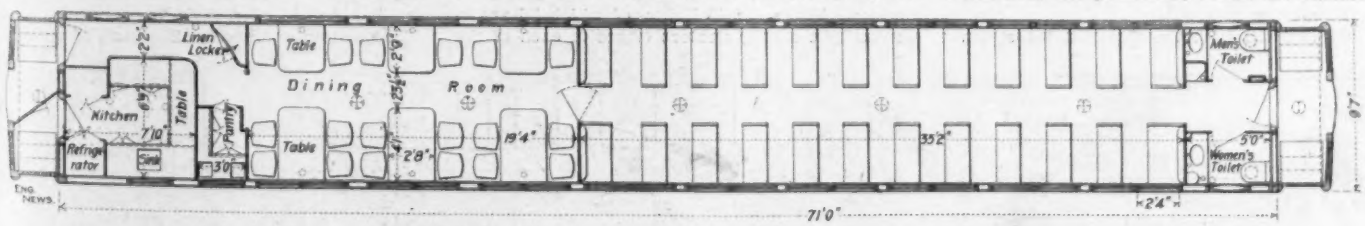
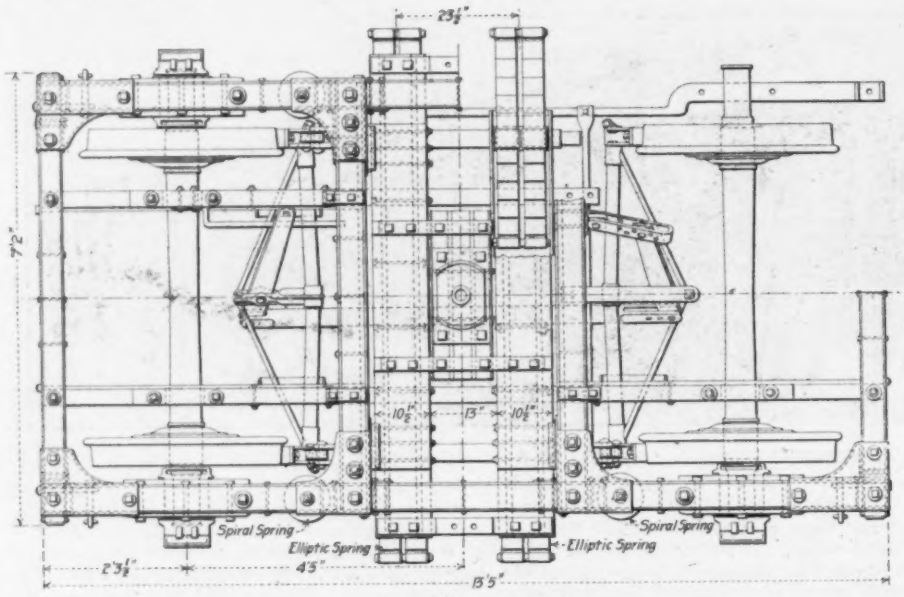
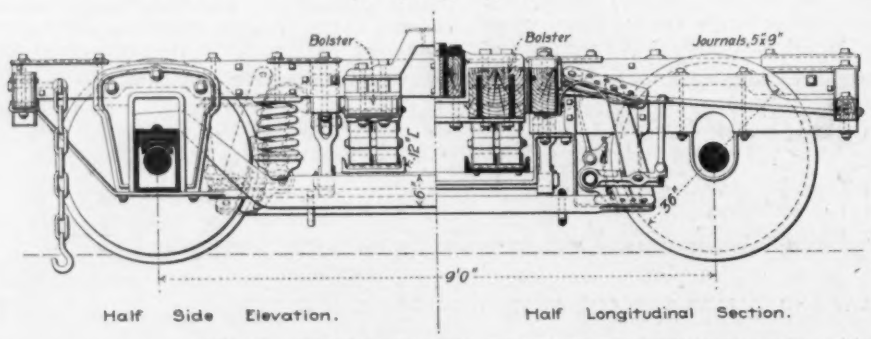


FIG. 3. PLAN OF COMBINATION DAY AND DINING CAR; CHICAGO, INDIANAPOLIS & LOUISVILLE RY.
Charles Coller, Master Car Builder, Lafayette, Ind. American Car & Foundry Co., Jeffersonville, Ind., Builders.

The car is mounted on four-wheel trucks, and was built by the Barney & Smith Car Co., of Dayton, O.

On the Chicago and Louisville day trains of the Chicago, Indianapolis & Louisville Ry., combination parlor and cafe dining cars are used. The dining-room, which has only two tables, is at one end of the car, and is connected with the parlor section by a corridor passing around the kitchen, smoking-room and toilet-rooms. The latest type of combination car on this road is a combination





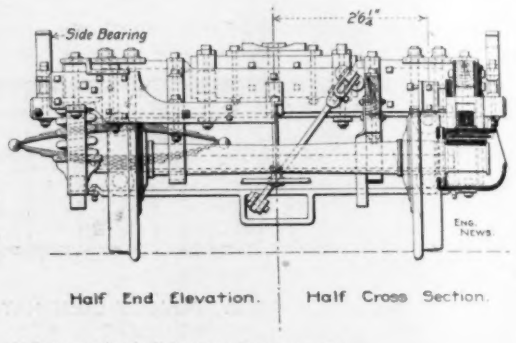


FIG. 4. DOUBLE BOLSTER TRUCK FOR COMBINATION CAR; CHICAGO, INDIANAPOLIS & LOUISVILLE RY.

day car and dining car, a plan of which is shown in Fig. 3. This was designed by Mr. Charles Collier, Master Car Builder, and built at the American Car & Foundry Co.'s works at Jeffersonville, Ind. The car is 70 ft. long inside, exclusive of the vestibuled platforms. The coach section has seats for 48 passengers, and adjacent to this is the dining-room, with 18 seats. At one end of the car is the kitchen, and the two toilet rooms are at the opposite end. The interior finish is of mahogany, with upholstering in green.

The car is fitted with the Consolidated steam-heating system, Pintsch gas lamps, and Sessions steel platforms (with C., I. & L. Ry. buffer attachment), and Trojan couplers. A special feature of the car is that although both long and heavy it is mounted on four-wheel trucks, which are specially designed for the service and are of the double-bolster type, shown in Fig. 4. These have four sets of elliptic springs between the bolsters and spring planks (12-in. channels), and four sets of spiral springs between the frames and equalizers. The center plates and side bearings are of the Baltimore ball-bearing pattern. Steel-tired wheels are used, of the McKee-Fuller make. The weight of the car is 104,600 lbs., of which the trucks represent 29,600 lbs.

The Louisville & Nashville Ry. also runs combination dining and day cars on certain routes. One of these cars is illustrated in Fig. 5, from plans sent us by the late Mr. Pulaski Leeds. The car is 65 ft. long over the body and sills. At one end are the toilet room and heater, on opposite sides. Next to these is the passenger section, 25 ft. 6 ins. long, with 14 high-backed seats, accommodating 28 passengers. Then comes the dining-room, 20 ft. 2¼ ins. long, with six tables, accommodating 24 persons at once. The kitchen and serving room take up the rest of the car, 18 ft. 3 ins., and a corridor affords communication past the kitchen to the other end of the car. There are four electric fans, two in each of the passenger sections. There are 49 lights, of which 36 are in six six-light ceiling clusters (three in each of the

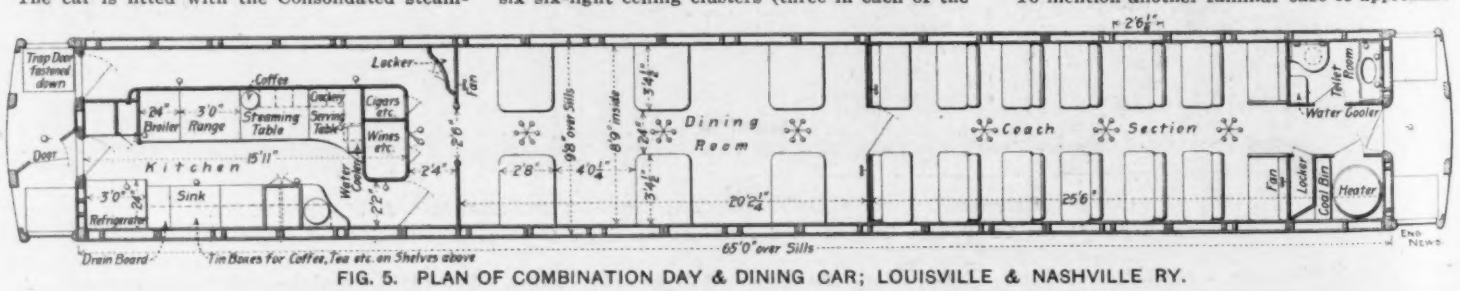


FIG. 5. PLAN OF COMBINATION DAY & DINING CAR; LOUISVILLE & NASHVILLE RY.

two passenger sections), while the others are in single and double wall brackets. It will be noted that while the car of the Chicago, Indianapolis & Louisville Ry. accommodates 48 passengers in the coach section and 18 in the dining-room, the car of the Louisville & Nashville Ry. accommodates 28 and 24 persons, respectively. The dining-room is, of course, for the use of all passengers on the train.

INACCURACY IN SOME ENGINEERING APPROXIMATIONS; FORMULAS FOR ELEVATION OF OUTER RAIL ON CURVES.

By W. D. Taylor,* M. Am. Soc. C. E.

A good deal has been said in Engineering News recently about the necessity of every engineer's using plain, well-considered and correct terminology in engineering papers; and a good many stereotyped expressions in use in numberless specifications have been condemned for good and sufficient reasons. The writer would call attention also to the fact that among engineers loose habits in diction are probably no more common than loose habits of thought. We engineers oftentimes acquire habits of using approximate methods where an accurate method would be no more difficult to apply. The old-time field railroad engineers are usually great advocates of approximate methods. The writer at one time was called on to approve the construction records of an engineer where the company the writer represented was to pay a certain annual interest percentage on half the cost of construction of a certain costly line for the privilege of joint rights in operation. The work had been laid off in a great rush and the cross-sections had not been made as often as was advisable in such rough country. When the construction notes were submitted to the writer he found individual stations whose contents had been calculated by the method of averaging end areas, which gave as much as 20% in excess of the true contents. There were two piers of first-class bridge masonry on this work, each of whose dimensions, as nearly as the writer can recall, were about as follows: Size under coping, 5 × 22 ft.; batter all around, 1-12; height, 80 ft. In calculating their contents the area under coping was taken and averaged with the area at the bottom of the pier. Let any engineer make the calculation for himself and assume the masonry to cost $12 per cu. yd., and he will find that the contractor would have been paid on these two piers alone more than $2,000 in excess of what was due had not a prismoidal correction been insisted upon. The method of averaging end areas is sufficiently accurate when used intelligently, but an engineer should know when his approximations do not approximate.

To mention another familiar case of approximation. Take the familiar formula connecting the superelevation of the outer rail with velocities on curves of different degrees:

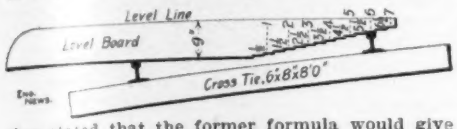
$$e = \frac{G\ v^2}{32.16\ R},$$

or, in more convenient form, $E = 0.00069\ V^2\ D$.

*Professor of Railway Engineering, University of Wisconsin, Madison, Wis.

The writer has been surprised at the unanimity with which some field engineers do not see that G in this formula is not the distance between the gage planes, but the distance between rail head centers, notwithstanding the fact that every field book and book on track on the writer's desk at this time which mentions the subject at all, such as Nagle, Webb, Wellington, Searles, Carhart, Tratman and Camp, plainly states or indicates that the correct value for this constant is the latter distance.

In March, 1902, the committee on track of the American Railway Engineering and Maintenance of Way Association submitted a report in which



they stated that the former formula would give "essentially correct theoretical elevations" when G was taken as 4.708 ft. In the discussion of the report the writer suggested that 4.9 be substituted for 4.7. To this suggestion the Chairman of the committee said: "I do not think that the committee would feel like accepting that amendment without working it out. The results worked out from the formula as they stand vary less than ½-in. for different speeds from results worked out by the *absolutely correct theoretical formula.* . . . ."

There was a table worked out to accompany the report connecting the velocity and the superelevation which gives elevations too small for the corresponding velocities by about 4%. The authority of this association now stands behind this incorrect table. Probably no trains will ever be turned over on account of the error in this table, but ought engineers be content to accept even an approximate table setting forth engineering truths which contains a constant error of as much as 4%, especially in such a case as this where a correct table is as easily calculated as an incorrect one?

The writer recently had submitted to him for criticism by a fellow engineer some most excellent (copyrighted) diagrams for use in putting in superelevation and transition curves, in one of which diagrams curves of different degree had been plotted from the second formula above, using superelevations in inches for ordinates and speeds in miles per hour for abscissae. But the constant used had been .00066 instead of .00069. In defending the use of the former constant, the engineer referred to said: "The trackman uses a 4-ft. 8½-in. gage, and the elevation is given to the nearest quarter of an inch for that gage." Now, that is precisely what the trackman doesn't do. If the trackman attempts to elevate one rail 6 ins. above the other, he makes the outer corner of the outside rail 6 ins. higher than the inner corner of the inside rail (see accompanying sketch). And these points are the same distance apart as the rail head centers and (except for slight inaccuracy on account of the coning on new wheels) as the centers of pressure of the wheel treads on the rail heads. Of course if the gage is widened either by or on account of curvature and 80-lb. Am. Soc. C. E. section rail is used, the constant 0.00069 itself is not large enough.

THE FREEZING PROCESS FOR CALKING A COFFERDAM

On page 127 of our last issue we briefly noted the use of the freezing process on a small scale at the large cofferdam of the Union Electric Light and Power Co., St. Louis. At our request the following particulars of that work, with a photograph illustrating it, have been sent us by Mr. W. V. N. Powelson, Assistant to the General Manager of the company named.

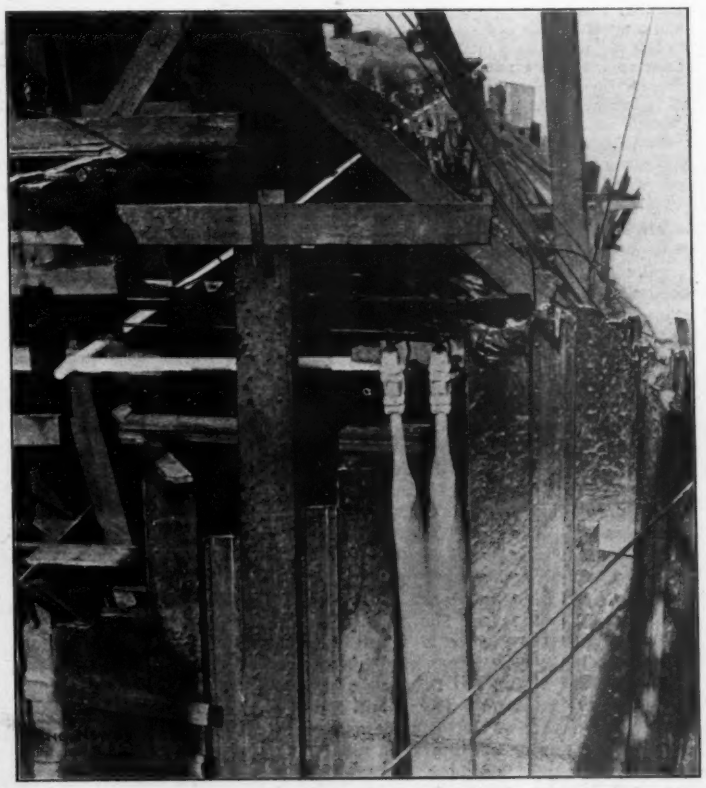
Replying to your letter of Feb. 9, the cofferdam referred to is of the interlocking channel bar construction made of steel piles 50 ft. long and driven down to bed-rock. In driving these piles for some reason they got out of line, and as the dam now stands this steel piling is lapped in two places with an opening anywhere from 2 to 10 ins.

At the time of the recent rise of the Mississippi River this dam, and the boiler-room behind the dam, had been excavated to a depth of from 10 to 20 ft. below the water level. The rising water washed out the dirt and other material placed around the part of the dam where the laps occurred and poured into the cofferdam and boiler-room excavations in quantities beyond the capacity of the pumps.

After the river had fallen somewhat, three pipes were driven down about 20 ft. into the mud between the piling at each lap. These pipes were ordinary 2-in. pipes fitted with a solid conical block welded in at the bottom to form a spike. From the St. Louis Refrigerating & Cold Storage Co., situated two blocks from the cofferdam, ammonia pipes were run, consisting of a ½-in. pipe for the liquid ammonia, and a 2-in. pipe for the vapor return. The ½-in. pipe was run down within a foot of the bottom of the 2-in. pipe, which was driven into the mud and suitable connections were made at the top. The liquid ammonia entering the ½-in. pipe passed down through it and into the 2-in. pipe and in a vapor condition into the return 2-in. pipe to the refrigerating plant.

Laborers were employed to pour water night and day for two days upon the pipes above the earth and water line. The result was that a coating of ice was formed around each 2-in. pipe forming a solid plug of ice which completely closed the laps in the dam.

Since the pipes were installed there has been a moderate rise in the Mississippi River and although the water is now some 2 ft. higher than the stage at which it broke into the dam, the work is now dry.



REFRIGERATING PIPES AT THE STEEL COFFERDAM OF THE UNION ELECTRIC LIGHT & POWER CO., ST. LOUIS, MO.

A ROLLING-LIFT BRIDGE OF UNUSUAL TYPE was put into service on the Newark Bay crossing of the Central Railroad of New Jersey on Sunday, Feb. 14. The bay is crossed by an old trestle bridge, some two miles long, in the middle of which is a swing-span. The latter has long been too light for the traffic, and is now being replaced by two distinct draws, each of which is a Scherzer rolling-lift bridge of 110 ft. clear span or 120 ft. between centers of piers. The two draws are back to back, with the operating house between them. They may be operated independently or together. One of the two spans occupies part of the channel of the present draw, while the machinery house and the other span are clear of the draw. The method of erection adopted to place the new draws without interrupting traffic is as follows: The machinery house and the span away from the present draw were the starting point. Concrete piers founded on piles were built underneath the old trestle, a rather difficult piece of work; the longitudinal girders carrying the machinery house and the rolling-track for the bridge were put in and the machinery house built. The span was, as usual with Scherzer bridges, built in raised position. It was completed, except for ties and rails and some of the heel bracing, by Feb. 14. Traffic on this day was diverted around the crossing by way of Newark. The heel bracing was put into place, that part of the trestle which occupied the future channel of the span in question was sawed off, and the span then was successfully lowered down on to the far abutment pier. Rails and ties were placed on the stringers and traffic was resumed by night. As soon as navigation opens this new draw will be used for passing vessels, the old draw will be closed permanently, and the other rolling-lift span will be erected as soon as its piers have been placed. A fuller description of the work will be given in our columns shortly.

THE WABASH RY. CANTILEVER BRIDGE over the Monongahela River at Pittsburg, Pa., is near completion, the first connection of the two sides having been made a few days ago. This bridge was described in a general way in our issue of November 20, 1902, but a fuller account of the work will be given at a future date.

THE STEAMSHIP "DAKOTA," for the trade from the Pacific terminus of the Great Northern R. R. to Oriental ports, was launched from the yard of the Eastern Ship Building Co., at New London, Conn., on Feb. 6. The "Dakota" is a sister ship to the "Minnesota," launched from the same yard in April, 1903. These vessels are among the largest afloat, being excelled in size only by a few of the latest additions to the White Star fleet, as reported in detail in our issue of Dec. 10 last.

TROUBLE FROM SNOW SLIDES on the Union Pacific Ry., in California, is reported in a news dispatch. It is said that snow sheds have been crushed at a number of places, and traffic blocked temporarily.

A SURVEY OF THE ALASKA BOUNDARY on the basis of the arbitration agreement reached last summer, is to be carried out jointly by the United States and Canada. Superintendent O. F. Tittmann of the U. S. Coast and Geodetic Survey and Dominion Astronomer W. E. King constitute the commission which is to conduct the survey.

THE PARIS METROPOLITAN RAPID TRANSIT LINE will, as an indirect result of the great tunnel holocaust a year ago, make a trial of the electro-pneumatic system of train-control developed by the Westinghouse Electric & Mfg. Co., of Pittsburg, Pa. It is stated 100 car-equipments of this type have been ordered.

THE TOTAL COAL PRODUCTION of the United States from the beginning of its mineral industry up to Dec. 31, 1902, is estimated, by the U. S. Geological Survey, at 4,800,000,000 short tons. That means that a pyramid built of this material as high as Pike's Peak (14,108 ft), would have for its base a rectangle 1.14 miles square. If the coal were spread out over the states of Rhode Island and Connecticut it would cover both of them a foot deep.

HARBIN, the commercial and administrative center of Manchuria, according to U. S. Consul Miller of Niuchwang, has grown in three years from a village to a city of 60,000 population, according to a census taken last October. The railway and administration employees constitute 11,000 of the population, and the Chinese, who occupy a separate quarter, number 40,000. The Russian government has expended about $15,450,000 on the city. At this place the Chinese Eastern Ry. branches from the main line to Vladivostock and runs north to Port Arthur. There is also water transportation for light-draft steamers up the Sungari River for 200 miles, while large steamers run down the Amur River, a daily service being established. It is a five-days' voyage to the junction with the Transsiberian Ry., where connection is made with trains for Vladivostock.

The country around Harbin is a great wheat growing district, and eight large flour mills are now in operation, which will be of great assistance in supplying the Russian armies in the East.

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In requesting change of mailing address, state BOTH old and new address; notice of change should reach us by Tuesday to be effective for the issue of the current week. The number on the address label of each paper indicates when subscription expires, the last figure indicating the year and the one or two preceding figures the week of that year; for instance, the number 325 *means that subscription is paid to the* 32d *week (that is the issue of Aug.* 10) *of the year* 1905; *the change of these figures is the only receipt, unless one is sent by special request.*

ADVERTISING RATES: 20 *cents a line. "Want" and "For Sale" notices, special rates, see pages* 26-27. *New copy for standing advertisements should be received ten days in advance of publication; new advertisements, Monday morning. The pages containing "Want" advertisements are held open until Tuesday, and those with "Proposal" and "For Sale" advertisements until Wednesday at* 11 *a. m.*

The remark quoted in a letter from Cincinnati in this issue that the Baltimore sky-scrapers were "destroyed by the flames like grass in a prairie fire," is matched by a report received in the same mail from a Chicago correspondent who writes: "The reports here said that 16-story buildings were reduced to ashes in a few minutes," as well as by the report of a newspaper correspondent on the spot who averred that "the steel-frame fireproof buildings in Baltimore burned up as if they were made of papier mache."

The contrast needs no emphasis between the picture set forth in these reports and that given in the photographs published in this issue, with the statement of a member of our own editorial staff prepared after inspection of these buildings.

Unfortunately, however, the general public forms its opinions, not from the careful statements of technical journals, but from the first hurried and sensational reports of the daily press, such as those quoted above. For many years to come, we presume, architects and engineers will find clients who are opposed to fireproof construction because, forsooth, "all the fireproof buildings in the Baltimore fire were burned up!"

The American Institute of Electrical Engineers did a notable thing in making its annual dinner, last week, a celebration of the 25th anniversary of the invention of the incandescent lamp and of the birthday of its inventor, Edison. It would be difficult to name another invention so small in physical size, out of which such large results have grown.

The most impressive feature of the incandescent lamp invention, viewed with our present knowledge, is that its originator and those associated with him should ever have been able to make it a commercial success.

The principle of the lamp was simple enough. It was known that carbon was a conductor of electricity, and was an unsurpassed source of illumination when heated to incandescence. It was also known that a supply of oxygen was necessary to combustion. If then a carbon rod were made slender enough to offer such resistance to the flow of current as to be heated to incandescence and were placed in a closed glass chamber from which the air was exhausted, it should glow without burning up, and an incandescent lamp would be produced.

But between the first incandescent lamp, created by the utilization of these well-known principles and the commercial success of incandescent lighting, there was a multitude of obstacles to be encountered; and it was in encountering these obstacles with full faith in ultimate success that the genius of the inventor and the courage of those financially associated with him were most notably displayed.

We may easily imagine some capitalist considering an investment in the new invention at its start, and exhibiting the new lamp to an expert with a request for advice. The expert, if thoroughly posted on the state of the industrial arts at that time, and if as conservative in judgment as an expert ought to be, would have had to reply about as follows:

> This is a very ingenious invention and it gives a very pretty light; but I must warn you that there are very strong chances that it can never be anything more than the mere scientific toy that all previous electric lamps, with the sole exception of the arc lamp, have been.
>
> In the first place the whole existence of the lamp depends upon a long, slender piece of carbon, which is nothing more than charcoal, as fine as a human hair. All our knowledge of charcoal goes to show that it is an extremely weak and fragile substance. It is true that the specimen you show me seems to have really surprising strength to resist jar, etc., without rupture, but the chances are strong that after it has been heated to incandescence for some time it will grow more brittle.
>
> Suppose, however, that this difficulty can be overcome, the point that appeals to me is: Can these lamps be made at a cost low enough to make them commercially practicable? The lamp consists of a glass chamber which must be specially blown, prepared and annealed and must withstand high temperature, while at the same time liable to exposure to cold drafts, and all the while under an external pressure of 15 lbs per sq. in. This glass globe must have a metal base attached with means for automatically connecting the current from the supply wires to the carbon filament; the carbon filament must be formed and put in place with nobody knows what expense and difficulty, and must be connected to wires made of platinum, a metal as rare and valuable as gold. Besides this, the interior of the lamp must be exhausted of air with a nearer approach to a perfect vacuum than has ever been attained, save in the most elaborate and costly experiments in a scientific laboratory.
>
> This is what must be done to produce each lamp, and if the lamp costs more than a fraction of a dollar, its commercial use will be exceedingly limited, if, indeed, it ever comes into use at all.

We do not think the above imaginary statement at all exaggerates the discouraging report that any conservative scientist would have felt bound to make on the incandescent lamp as first invented. Yet these difficulties and a host of others which we have not enumerated have all been surmounted, and we buy to-day an incandescent lamp, to whose creation all the resources of inventive genius and scientific investigation have been taxed, for little more than the cost of a common glass lamp chimney!

As for the unforeseen results that have grown from the commercial perfection of this little bulb of glass enclosing a carbon filament, it would take pages to record them. It has created new industries by dozens and hundreds. It has furnished a profitable field for the investment of capital measured in hundreds of millions of dollars; it has set a thousand cataracts at work for the benefit of the race; it has made a revolution in the comfort and safety of ocean travel and, indeed, of all travel and transport by water.

Let us not forget, too, that this vast range of new industries have given employment and opportunity to tens of thousands of civil engineers, mechanical engineers and mining engineers, as well as to the electrical engineers in whose particular department of the profession the development first began. It seems to us an admirable illustration of the interdependence of the different branches of the engineering profession.

A recent news item from an Eastern city states that in a building in course of erection in that city a section of floor on which were piled several tons of building material gave way and dropped through two or three floors below to the basement. There is nothing in such a piece of news that need surprise builders or building engineers. It is common practice in building work to use completed floors as storage spaces for material; brick, cement bags, tile, lumber and whatnot are brought in and piled on completed floors or floor sections until needed in the rear under of the work. The character of the floor, its calculated strength, its age or condition, etc., are rarely given special thought. So it happens that a floor designed for a safe total load of 200 or at best 150 lbs. per sq. ft. is made to carry a pile of brick or cement equivalent to a load of double that amount. The mortar in the joints of the floor tile, or the cement in the concrete constituting the floor, may still be too soft to give the floor its full designed capacity; yet the floor is thoughtlessly loaded in such a way that its margin of safety is seriously encroached on and sometimes exceeded.

Failures of floors or parts of floors resulting from such conditions are not exactly rare, though they are seldom heard of. When they occur the building is still in the hands of the contractor, on whom the loss falls, and he is generally disposed to keep the accident from the knowledge of others as far as possible. Fortunately loss of life in these accidents is rare. The collapsed section of floor is rebuilt and the failure is forgotten.

A failure, however, is not the only or even the most serious result of overloading such as has been described. The unknown and unknownable margin of safety in the freshly-completed floor is in the majority of cases sufficient to prevent a direct collapse. But before the point of failure is reached the partially-hardened mortar or concrete is stressed to such an extent as to deform it, or at least to derange its set and give rise to points of weakness in the construction. Or the load piled on the floor may arch to some extent and relieve the floor as the latter deflects under the excessive load, thus preventing collapse, but giving no guarantee that the strength of the floor is not materially impaired by the deformation. The resulting weakened condition is practically beyond detection, and the floor is put into service with its strength reduced below that which the construction was planned to have.

Careful watching to guard against overloading of a floor while the building is being erected suggests itself as the remedy, unless the floor is figured for particularly light loading, in which case it is worth consideration whether an increase in the figured loading ought not to be made to provide for erection stresses greater than those planned for and greater than the stresses to which the finished structure will ever be subjected. In the case of a tier building, the upper floors are generally figured for a very light live load, corresponding to the light service to which these floors will probably be exposed. It requires only a small pile of brick or cement to exceed this live load, and then the condition of overloading is reached even though the floor may have already attained its full strength. If the floor be not designed with a view to these loads, then strict inspection should be maintained to the end that no floor section is overloaded with building material during construction.

A recommendation for an increase in the Corps of Engineer officers of the U. S. Army was made by Gen. Gillespie, the late Chief of Engineers, in his last annual report, and this recommendation has now been renewed by his successor, Gen. Mackenzie, and by Secretary of War Root, prior to his retirement.

This request may seem surprising at first, in view of the fact that an increase in the corps was made only three years ago; but Gen. Gillespie's report makes clear the reason why the demands on the officers in charge of river and harbor work are greater now than ever before. We quote the following extract from the report:

> The act of February 2, 1901, increased the organization of the Corps of Engineers from 127 officers to 160 and the enlisted strength from one battalion to three battalions. The increase of officers was made up of 5 captains, 10 first lieutenants, and 18 second lieutenants. The extra battalions require the exclusive service of the following officers: Two majors, 8 captains, 12 first lieutenants, and 8 second lieutenants; the net effect of the increase was the loss of 7 officers from the grades which are most effective for duty other than with troops.
>
> While the organization of the Corps of Engineers provides for 160 officers, law, orders, and regulations now provide for not less than 110 positions that must be filled by officers of the Corps of Engineers, independent of duties relating to construction of fortifications and works of

river and harbor improvement. A further reduction in number of not less than 16 must be considered for officers sick, on leave, or undergoing instruction at schools of application. As a result there remain but 27 officers specially available for construction work in not less than 54 engineering districts throughout the country.

There is a general impression among civilian engineers that the Corps of Engineers is chiefly occupied on River and Harbor works. The above extract shows that this is far from being the case. Fortification works, the construction and repair of lighthouses, the construction of public buildings, the municipal engineering of the national capital, the improvement of Yellowstone Park, the building of dams for impounding mining debris in California and the survey of the Northern Lakes are among the other duties which must be discharged by the Corps; and it is to be noted that the constant tendency is to increase the amount of work assigned to this organization.

As the case stands, according to General Gillespie, there are only 22 officers available to take responsible charge of River and Harbor works involving an annual expenditure of perhaps $20,000,000 to $50,000,000. We doubt whether any engineering work in this country is carried on by so small an engineering force in proportion to the amount of money expended.

Of course these officers have civilian assistants, but these men are notoriously underpaid, considering the amount and character of the work placed upon them, particularly in the higher grades. While many very efficient and competent engineers are to be found among these so-called "Assistant Engineers," the tendency is all the time for the best men to leave the service for some other field where a stone wall is not built across a man's path toward promotion to higher responsibilities and a larger salary.

General Gillespie's recommendation was that the Corps of Engineer officers be increased from the present number, 160, to 185; as this is the very least that can properly supervise the work for which the Corps is now responsible. It is to be hoped that Congress may see fit to act upon this recommendation. The government pays out money in very few places where a larger return is given than it receives in the services of the Corps of Engineers; and it is not only unfair to these men to load them with more work than they can well perform, but the poorest sort of economy besides.

When a man is given more duties than he can properly attend to, he must neglect some part of them. In the case of such work as River and Harbor improvement, an engineer will naturally and properly attend first to the quality of the work done under his charge, and if he must neglect something, he will let it be the matter of economy.

We do not mean by this that he will neglect supervision of work under construction; but the greatest opportunities for economy are in the original design of works. It is often possible, by taking time to study over a completed plan, to find many ways of saving money either in first cost or in the cost of maintenance, without sacrificing quality in any way. It is this class of economies which the engineer whose employer believes in economizing in engineering expenses is often obliged to neglect.

A possible menace to the public health of some 90 per cent. of the population of Massachusetts now exists in the shape of Senate Bill No. 87, recently introduced in the Massachusetts legislature. To a large extent the proposed bill would annul the present powers of the State Board of Health to make regulations for the sanitary protection of the public water supplies of the State, since those regulations would be subject to alteration or annulment, on appeal, by juries in county courts. The verdicts of such juries, it is true, would not be effective unless confirmed by the county courts, but the effect of the whole act is to enable any aggrieved person to substitute county juries and judges for the State Board of Health. Other objectionable features of the proposed act are stated in our report of the February meeting of the New England Water-Works Association, elsewhere n this issue, and the full bill, with a detailed explanation of its significance, can be obtained from a committee of the New England Water-Works Association, of which Mr. Willard Kent, of Narragansett Pier, R. I., and Charles W. Sherman, Division Engineer Metropolitan Water-Works, Boston, Mass., are members. It is to be hoped that the bill will not be passed, since it would be a backward step for Massachusetts and a stumbling block to sanitary progress elsewhere.

BALTIMORE'S NEED OF A WELL-CONSIDERED MUNICIPAL PROGRAM.

A rare combination of disaster and opportunity confronts the people of Baltimore. That they will prove superior to the one and seize the other was indicated by the fact that the news, "Baltimore in Ashes" had hardly been sent over the country before it was followed by the announcement that citizens and officials were alike resolved that out of the smouldering ruins there should speedily rise a new city far surpassing the old one. Those words of determination, uttered in the midst of overwhelming calamity, have been accepted at their face value and the country now confidently awaits their fulfillment.

Baltimore's opportunity for increased municipal greatness is largely due to the fact that heretofore the city, like most municipalities, has developed without regard to any well-defined plan. Nature and man have combined to swell the population and industry of the city, but there has been comparatively little intelligent, concerted direction of the growth of the whole municipality and the means for supplying its wants.

A well-considered municipal program is the first and greatest need for the rebuilding of Baltimore, By a municipal program we mean a broadly conceived plan for the future development of the city which shall take into account the needs of all branches of the municipal service, as well as the financial ability of the city to meet them, and shall indicate the relative importance of those needs and thus the order in which they should be provided. We are pleased to learn that the necessity or desirability for some such action was promptly recognized by Mayor McLane and others, but there is grave danger that the pressure from all quarters for the speedy rebuilding of places of business will be yielded to before any well-considered plan can be matured. Moreover, herein lies a danger that the city will not only be rebuilt on essentially its old lines, but also that flimsy and readily inflammable structures will be erected. The latter danger will be all the greater because of the urgent need of providing new business houses. Only a thoroughly competent and fully manned building department will be able to resist the pressure due to motive of short-sighted haste and economy and to insist rigidly on the adoption of all reasonable safeguards against fire.

As for providing a municipal program, that should be entrusted to a most carefully chosen body of representative citizens, acting with the heads of the various municipal departments, and provided with full authority and ample funds to employ engineering and other experts, not forgetting those competent to give advice regarding the artistic and other higher phases of city development. Such a commission as has been suggested should not confine itself simply to making good, or rather making better, the losses entailed by the recent fire. It should consider and plan for all the needs of the city for years to come, as well as the financial problems involved in meeting those needs. It should bear in mind the financial strain to which the city will be subjected in its proposed regeneration and guard most carefully against the starvation of some city departments in order that others may be well provided for.*

To outsiders it would seem that the first two considerations in formulating a municipal program for Baltimore would be (1) widening and straightening streets, and possibly providing new streets, some of which might, perhaps, be on the radial or diagonal plan, at the same time having in mind the desirability of public squares and other open areas to furnish breathing spaces, to afford opportunity for esthetic effects and to assist in checking the progress of some future conflagration, (2) fire resisting construction. These two measures, as all can see, need the promptest attention, since any readjustments of street and building lines should be made before rebuilding is begun and since the erection of buildings which will invite another conflagration should be prohibited from the outset.

The experience of Baltimore has illustrated afresh, what every well-informed engineer already knew, but what the general public is so slow to grasp, that no human ingenuity can devise a building of reasonable cost and utility which will withstand unimpaired the attacks of such a conflagration as raged last week in Baltimore. On the other hand, if all buildings in the business section of a city were reasonably fire-resistant, as they might be at moderate cost, large areas could not be fire-swept and utterly destroyed. The fire-resisting power of modern construction is well attested by some of the buildings standing in Baltimore to-day. To draft, enact, and, above all, to enforce the needed ordinances for fire protection in the rebuilding of Baltimore would require as much if not more moral courage than has yet been shown since the fire; and herein lies a fine opportunity for the new Baltimore to show its superiority to the old city.

A crying need of Baltimore for some scores of years has been a system of sanitary sewers. That an American city with a population of more than 500,000, standing sixth in the list of the largest cities on the continent, should still be without sanitary sewers, and yet should have spent hundreds of thousands of dollars for storm sewers, has long been a mystery to sanitarians. At first thought it might appear that the city would now have to postpone indefinitely the introduction of sewers, but what a shame and what a business handicap and financial loss it would be to rebuild Baltimore and have neither sanitary sewers or the prospect of any with which to connect new buildings!

Every city of the size of Baltimore should provide itself with subways for pipes and wires in order that all its underground furniture may at all times be accessible for inspection, repairs and new connections without injury to pavements or interference with street traffic. The construction of electrical subways was begun some years ago in Baltimore, but some of the private companies using poles and wires have been very backward, to say the least, about availing themselves of the electrical subways provided by the city. Mr. Charles C. Phelps, Engineer of the Subways, is reported in one of the Baltimore papers as having made the gratifying announcement that the city Electrical Commission will not permit the erection of a pole or the stringing of a wire, except trolley wires, in the burned district. Mr. Phelps is also reported as having said:

> The Electrical Commission met yesterday and decided upon this course. Most of the poles in the territory swept by the fire are already down. The rest are being taken down by squads of men under my direction. They are being cut up as fast as they are taken down and carted away for distribution among people who need wood.
>
> Under the direction of the commission our men are isolating the burned district from all live wires, and absolutely none for even telegraph, telephone or lighting purposes will be permitted to be strung. In this district the city conduits are completed and ready for the reception of wires of all companies. Most of the companies have their cables already on hand, while those that have not can easily get them. No permits for poles will be issued. They are down now and they will stay down. The commission has decided upon this course and will stick to it.

Telephone wires were reported underground in the latter part of 1901; but the tangled mass of wires in the streets of the burned district were one of the striking features of the ruins and must have been during the fire, and in cleaning up the wreckage will doubtless continue to be, a serious nuisance.

In view of all the pressing needs of Baltimore it may seem as though subways for pipes should hardly be mentioned at present, but this and many other essentials to full municipal devel-

*We understand that Mayor McLane has already appointed a large advisory committee of citizens to aid the city officials in deciding some of the questions now pressing for immediate settlement. This committee, we fear, is too large and too general in character to work out such a municipal program as we have in mind.

opment may properly be considered now, for nothing so conduces to the attainment of an ideal condition, whether by a city or by an individual, as having an ideal to struggle for and measure one's success by. The great trouble with our American cities is that they have no well-defined ideals and that, for the most part, their ill-defined ideals are unworthy of the twentieth century. If the city of Baltimore, immediately after its great disaster, should establish for itself a municipal ideal, a consistent plan for future development, then it would have already taken a most important step towards fulfilling its brave prophecy that a greater city would rise from its ashes.*

LETTERS TO THE EDITOR.

The Lay-out of the Moline Shops of the Chicago, Rock Island & Pacific Ry.

Sir: In connection with the new shops for the Chicago, Rock Island & Pacific Ry., illustrated in your issue of Feb. 11, it is not inappropriate to call attention to the fact that in August, 1902, Mr. Alfred Brotherhood and myself were employed by the Rock Island Company to advise them in regard to the new shops, selecting a site, making a general lay-out, general tool list, general drawings of the buildings and a general specification. In October, 1902, Mr. Brotherhood and myself selected as the site for the new shops the one actually built upon, at the same time submitting a general plan for these shops. In December, 1902, the remainder of the work was completed and a drawing showing a general lay-out of shops, yards, etc., was submitted. Respectfully,

Geo. Hill.

100 Broadway, New York, Feb. 13, 1904.

Schools of Railway Engineering.

Sir: In your issue of Feb. 11th you printed an article in your editorial column relating to Railway Engineering Schools.

I wish to state that the school to be established at McGill University and the school already in operation at Cornell University differ radically.

The school at Cornell is a mechanical engineering school, and deals directly with the motive power department, including shops, rolling stock, and equipment. When this school was established it was decided that the civil engineering college was adequately equipped to give instruction in the department of maintenance of way, bridge building, and in fact all branches of engineering which lie directly in the province of the civil engineer.

The school was named the school of railway mechanical engineering, for this reason, This school has already solved the problem as you suggest it should be solved in the last paragraph of your article.

The course which practically begins in the senior year, consists essentially of a study of existing shops and shop methods, and open discussion of good and bad features, locomotive design, car construction, the air-brake, the existing link motion and limitations, the slide valve, and, in fact, each detail of the locomotive, good and bad features being dwelt on.

Each student intending to take the course is urged to work at least one summer in an erecting or repair shop, and another summer in the drafting room. A post-graduate course is offered to students continuing on the same lines and dealing more thoroughly with the subject in hand. Yours truly,

H. Albert Rogers.

4811 Regent St., Philadelphia, Pa., Feb. 15, 1904.

A Notable Fire Test of Different Systems of Fireproof Construction.

Sir: The problem of fireproof construction is one which concerns the interests of the engineering world in a large measure and to-day it is brought into the foreground with great emphasis by the conflagration in Baltimore, where, according to the newspaper accounts, skyscrapers were destroyed by the flames like grass in a prairie fire. Under these circumstances it will surely be of great interest to the architectural and engineering world to know that a fire test will be conducted under the auspices of the University of Cincinnati to determine the relative value of ferro-concrete and fireproof steel-frame construction in the erection of large buildings.

The test will be made on two buildings, one of ferro-concrete, to be erected by ferro-concrete contractors, and another of fireproof steel-frame construction, to be erected by contractors using this method of construction. The fireproof steel and the ferro-concrete interests are each represented by a distinct committee. Each committee will make recommendations as to size, details and requirements of buildings to be tested.

The advisory committee, consisting of Architect Harry Hake, Chairman; Architect Harvey Hannaford, City Building Inspector; Charles Tooker, a representative of the National Board of Underwriters, and Professor C. W. Marx, of the University of Cincinnati, as expert engineer, will pass upon these various details, etc., and prescribe the size, requirements and details which each interest must comply with in the erection.

Cincinnati is the very center of the ferro-concrete interests and it is the intention to make this test as exhaustive and conclusive as possible to determine the relative fire-resisting properties of both constructions.

In order to have the test as impartial as possible the above committee was elected by the organization, and it was deemed best to have the university take charge of same. As a member of said committee I will be pleased to receive suggestions by letter from all persons interested in this effort to determine the relative values of these different types of fireproofing as to their resistance to fire and water. Very truly yours,

C. W. Marx.

University of Cincinnati, Cincinnati, O., Feb. 11, 1904.

The Object and Needs of the American Society of Civil Engineers.

Sir: I take it that those who seek membership in the American Society of Civil Engineers do so for two principal reasons. One is to increase their professional knowledge by personal intercourse with other members, and through the papers and discussions presented at meetings. Obviously the greater and wider the facilities offered for this, the better. While engineering is constantly becoming more and more specialized, yet more and more the specialist finds it necessary to familiarize himself with other branches which interlace with his own particular work. Not to the same extent that specialists in these branches require, but just in that general way best brought about by closer association with others directly connected with those branches. No one can doubt that as far as this object goes, it would be greatly advanced by having the headquarters of the mechanical, electrical and mining specialists brought within easy reach of our own headquarters, in a common building, where interesting papers were read and discussed, and conversation could be had upon these specialties. In other words, these increased facilities would tend to increase and not to decrease our range of professional information, and our juxtaposition to the other societies would, in so far, be beneficial and not detrimental. In this respect nothing could be lost and much might be gained by co-operation.

The other reason is, the professional standing which membership in our society gives. Different technical schools confer, under varying conditions, the degree of "C. E." on their graduates, but it is universally recognized at home and abroad that the only valid title to this distinction is the authority to write "M. Am. Soc. C. E." after one's name. Now the distinction which this title conveys depends entirely upon the qualifications exacted before conferring it, and the professional and moral standing of the officers of the society who adjudicate these qualifications, and upon nothing else. The locality of the seat of the society, whether in 57th or 39th St., whether in self-contained or semi-detached premises, has nothing to do with it. If we select men of high professional and moral standing for our officers, and maintain a high standard of qualification for membership, there is no fear that the dignity and distinction of our title can be sullied by moving from our own little house to the monumental quarters offered us. The danger to be avoided comes from within and not from without. No one can lower our standing but ourselves. I refrain from dwelling longer upon this topic.

As to the adequacy of our present house, it is asserted that it is abundant for all purposes except on one day of the year, that of the annual meeting. But that is precisely the one day of the year when the building, with its auditorium, reception rooms, banqueting hall, etc., come into full play; the very day, par excellence, when it is essential that they should be adequate. It is the only day when a large proportion of members ever see the home of the society at all, and yet we are told that on that one supreme day we can easily "hire a hall" and go from one locality to another for our proceedings, putting the society on wheels, as it were, without any inconvenience. This is, clean cut, the argument of the member resident in the city of New York who very often does not attend annual meetings at all.

We want greater opportunities for broadening our field, and we need more room for the one day of the year when members from all over the country gather the thickest. Mr. Carnegie's splendid offer meets both needs. Moreover, it comes in the nick of time. What course of action in regard to it is the sanest and most logical?

E. Sherman Gould, M. Am. Soc. C. E.

Yonkers, N. Y., Feb. 12.

The Union Engineering Building.

Sir: My attention has been called to an incomplete statement, or omission, in my communication relating to the proposed Union Engineering Building, in last week's Engineering News, without which my meaning is obscure and the statement, in one item, might prove incorrect.

It was stated that the American Society of Civil Engineers would, if it invested in one-fourth the bonds to be issued by the executive corporation, be entitled to interest on them at 4%. It was intended to say, when speaking of the probable expense of the society, that it would also be assessed by the corporation for its share of the interest upon the bonds, and the credit and charge would thus cancel each other. Each of the other societies would be in the same position if each held its share of the bonds, but each would be assessed for its share of the interest whether it held bonds or not; that is, if some of its share of the bonds were held by outside persons. If any one society should default in paying its share of interest, the remaining societies would have to make up the deficit, just as in the case of default in paying operating expenses. The *possibility* of such an occurrence cannot be denied, though its *probability* is very remote. It is, however, one of the contingencies that must be considered.

S. Whinery.

95 Liberty St., New York City, Feb. 13, 1904.

Sir: I am unalterably opposed to seeing the scalp of the American Society of Civil Engineers dangling at Mr. Carnegie's charity belt, and I hope a majority of the members share the feeling. He offers us a one-fourth interest in a house on conditions; we now own a whole house with no conditions attached, which we are expected to sell in order to acquire the aforesaid one-fourth interest. The Society has achieved a position and accumulated property through the splendid efforts of its successive Boards of Direction; a strengthening of this position and the further accumulation of property, with title resting in the Society, may be safely left in the same hands.

If a merger be considered wise, let the Board of Direction take the initiative, not an outsider. From a "Carnegie suitable Union Building" to the "Carnegie Society of Un-American Engineers" is but a short step.

I have the utmost sympathy for anyone afflicted with alcoholism, money or other serious disease, who is honestly striving to abate the spasms, but I do not believe the American Society of Civil Engineers can safely aid Mr. Carnegie in separating himself from a million or so. We are indebted to him for a very good article of steel, which we use; he publishes a little book, which we also use on occasion—let us not then, even at his urgent request, increase our obligations. A gentleman cannot accept tips; a society of gentlemen cannot collectively accept tips. We are engineers, not waiters. Yours truly,

Edwin H. Warner, M. Am. Soc. C. E.

National Bank of Commerce Bldg., Tacoma, Wash., Feb. 4, 1904.

Sir: I hope that my brother-members will not take amiss this appeal for careful consideration of the "Statement of the Board of Direction in the matter of the Proposed Union Engineering Building" before casting their ballots.

As at first presented, the proposition did not commend itself to the judgment of many of us, possibly because of the inadequate manner in which it was brought to our attention. But since the Board of Direction was authorized to prepare a statement to accompany the resolution for a letter ballot, the suggestion of Mr. Carnegie has taken an ampler and a more definite form. It is now virtually a project for the establishment of a university in which the application of science and of the useful arts to the uses of civilization by the engineering professions may be worthily demonstrated. If carried through successfully, it will not only be a monument to the liberality and the foresight of the man who has made it possible, but also to the spirit of enterprise which has elevated this country to the first rank of nations.

We cannot hope to vie with the great nations of Europe in picture galleries and museums whose claims to our admiration rest upon deeds of the past, but we may justly be proud of what American engineers have done in controlling the forces of Nature for the future welfare of mankind. Their masterpieces cannot be contained within four walls. They are spread over the whole earth. But the records of their glory can be housed within a single building, on the scale that is here proposed. Under the same roof that shelters them may be collected and systematized the growing mass of information on engineering subjects published in many languages and places, which would be otherwise unavailable to the members of any one of the societies. It would also be, in a certain sense, a professional arena in which the latest ideas in the several branches of engineering would be discussed and where the wheat would be winnowed from the chaff by men recognized as leaders in their specialties.

Let us keep one fact clearly in mind: that this is not a question of amalgamating the American Society of Civil Engineers with other engineering societies, but of closer association for their mutual welfare. Our society is what its members have made it and so it will continue. We

*The general aspects of the need for municipal programs were set forth in somewhat more detail in an editorial published in our issue of July 17, 1902, entitled "A Municipal Program Essential to Symmetrical and Continuous City Development."

have nothing to fear as to its prosperity or as to the estimation in which it is held, either at home or abroad, from propinquity to the other societies.

The possibilities that this project offers for securing the best results with the least expenditure of time and labor ought not to be lightly set aside. Think of its value to the younger members of the engineering professions in this country and think, too, of the impression which such a structure would make upon foreign visitors. To them it would seem a fitting symbol of the place which these professions hold in the estimation of the American people. Nor should the advantages be underestimated that are to be derived from the proximity of the Engineers' Club. The opportunities for social amenities between members of the several branches of the profession must work toward a closer affiliation among those who by separate avenues approach a common goal.

The arguments for and against the proposition are so fairly stated by the Board of Direction that nothing more is needed to enable us to vote intelligently upon it; nothing except to approach it with an open mind. Those of us who in the course of years have passed beyond the active practice of our profession, and who have gained about all that we might expect to derive from such a project, should consider what it offers to our younger brethren and, in casting our ballots, should keep their interests foremost in our minds. Nor should we forget that narrow views on important questions rarely prove to be correct.

If in your opinion this project be a worthy one, do not fail to do all in your power to make it a success; for, if this opportunity be lost, another like it may never occur again.

Very truly yours,

H. S. Haines, M. Am. Soc. C. E.

New York, Feb. 10. 1904.

Sir: The decision to be arrived at at an early date concerning the acceptance of Mr. Andrew Carnegie's generous offer to give a spacious building in New York City for the joint occupation, first, of the four main Engineering Societies, and, second, of such other allied scientific organizations as may need and can be furnished with accommodation, marks a period of vital importance to the engineering profession in this country. That decision will unquestionably be far wider-reaching in its effect than was supposed when the subject was first broached. It then and now, so far as the actual transaction on its face is concerned, meant and means only a joint ownership in and occupation of a conveniently arranged building, requiring no closer union of the different societies making up the holding corporation, than required as stockholders in a corporation. The arguments for and against such joint occupation of a building are simply those of details of ownership and management, which can surely be satisfactorily arranged on a suitable business basis.

The acceptance or declination of Mr. Carnegie's proposition by any one of the individual societies to be named in the deed of gift is to a great extent also purely a business consideration; simply whether the additional convenience to be secured in occupying the proposed building and securing such a large addition to its capital is or is not worth some slight sacrifice of identity. But there is a much more important question raised: that is, is not the time fast approaching when it will be desirable to abandon much of the narrow conservatism of the past, and taking joint occupation of the proposed building as the start, look to the ultimate amalgamation of the different societies and the organization of one great American Engineering Institution. Such a society may well have sections composed of those members particularly interested in special lines of engineering work, and branch headquarters in many places, but one central executive government.

Though to-day it is more the custom for "civil engineers" to be called into consultation on the broadest questions of corporate management and great constructive works, while engineers having more specialized training are not called in until questions of detail are to be considered—it is not improbable that in years to come the training of all engineers may be such as to primarily fit them for the breadth of duties and secondarily for the special fields.

Opposition to such union is largely on the score that some men are admitted to membership in some of the societies who are practically not engineers in the broad sense of the term. Unquestionably this is true, but admission to the great Institution can be set at a high standard, so that new members will be men of proper attainments, and in course of years those of low grade now on the rolls of individual organizations would either become worthy, by reason of continued experience, or drop out because out of keeping with their surroundings, or in course of time will surely drop out by death.

The medical profession many years ago was face to face with a proposed wide differentiation of practice, with the probable establishment of many different organizations; but fortunately wise heads controlled, and there is to-day but one medical profession, though with probably a much wider line of specialists than there is ever likely to be among engineers. There are probably some who believe that there is room in this country for a number of strong, separate engineering societies. About forty years ago there were many people who thought there was room in this country for many different governments.

The whole trend of progress during the last half century has been towards centralization and consolidation; and when honestly carried out, the general wisdom of such a course has been proven.

According to the amended present scheme, it would seem that the decision of the American Society of Civil Engineers (unless Mr. Carnegie should again modify his proposal) will determine the fate of the joint engineering building. Therefore, there is a moral obligation on the part of that Society if the joint building promises to be specially beneficial to the other societies, of aiding them to secure such benefit, if it would not result in seriously handicapping its own work. There would seem to be only one possible handicap in such a course, and that is the slight loss of prestige, perhaps, in not being able to point to its own separate building on a fine wide street in a central location, and to come in contact with some men not properly fitted for membership in such a society.

I hope personally that the American Society of Civil Engineers will waive petty matters and some of its conservatism, and will decide to join in the present plan, provided the details can be arranged on an equitable, sound business basis; and that the building be located where it shall be deemed most desirable, even if on another site than the one at present selected, and further, that the American Society of Civil Engineers shall lead the way in the organization of one great American Institution of Engineers. Yours truly.

Louis L. Tribus, M. Am. Soc. C. E.

New York, N. Y., Feb. 9, 1904.

The Work of American Chemists in the Investigation of Portland Cements.

Sir: I am very much astonished to read in an article, entitled "Opportunities for Chemical Engineers in the Ceramic Industries," reprinted in your journal for Jan. 14th, from a bulletin of the University of Wisconsin, certain statements by Edward Orton, Jr., in regard to the Portland cement industry in the United States which are quite erroneous. They are the more remarkable as coming from a source which would naturally be supposed to be more conversant with the subject than appears to be the case.

Mr. Orton states that "a good cement should not vary more than one-half of 1% in alumina, 1% in silica and 1% in lime." This admits of two interpretations. He either intended to state that good Portland cement, in general, should not vary beyond the limits given, or that a cement produced from any mixture of the same raw materials, should not exceed these limits. The first proposition is, of course, absurd, especially in regard to alumina. The second proposition is incorrect as the lime may vary quite widely according to the temperature attained in burning the clinker. With the same raw materials, as in the Lehigh Valley, larger differences in silica and lime in the finished cement are found depending upon whether the clinker is burned at a high or low temperature or quickly or slowly; at one plant a high temperature being found most satisfactory or economical, and at another a much lower one according to the local practice. The lime must, of course, be kept at a lower percentage with the lower heat. The degree of fineness to which the raw material has been ground also has a marked influence on the amount of lime which can be successfully used, the finer the raw material, the higher is the percentage which can be maintained and still produce a cement of constant volume.

If Mr. Orton intended to convey the meaning, which is not apparent, that at any one plant with any fixed method of burning the silica, alumina and lime should not vary beyond the limits given, he hardly makes them narrow enough.

Mr. Orton next asks the question: "What is Portland Cement?" and replies: "Accurately speaking we do not know what it is," and adds, "we have produced scarcely a single investigator who has contributed anything noteworthy to the knowledge of cements," and "it is a significant fact that scarcely a single contribution of importance to our higher knowledge of cement has yet come from America." I must interpose a most emphatic denial to all these propositions.

What Portland cement is has been recently very conclusively settled to any one having a knowledge of physical chemistry and the theory of solutions, and this has been done on this side of the water and not abroad. The work having appeared only in German has evidently escaped Mr. Orton's notice.

Of previous investigations in the United States, it may be said that in October, 1897, the Messrs. Newberry presented before the New York Section of the Society of Chemical Industry, a paper on "The Constitution of Portland Cement," of which London "Engineering," in its issue of Dec. 24, 1897, spoke as follows:

> No piece of work on the chemistry of cement comparable with this by the Messrs. Newberry has appeared within the last ten years, and we congratulate the authors on the sound method, well directed industry, and clear reasoning which are its characteristics.

How this paper has been regarded by Continental investigators appears in a review of the literature of the subject of the constitution of Portland cement by the writer which is now appearing in "Cement." It is true that it has been criticised and that theories of a different nature have been deduced from the facts advanced; but no one since the time of Le Chatelier, 1887, has contributed so many novel data based upon the actual preparation of synthethic products as the Newberrys have done. Their conclusions were that "the essential constituents of Portland cement are tricalcium silicate with varying proportions of dicalcium aluminate," and this simple conclusion stands confirmed to-day, with some slight modifications and with some further development and elucidation, by elaborate investigations carried out in the last three years by the writer on similar lines, except that the introduction of the microscope and petrographic methods has made it possible to explain the physical constitution of clinker more fully.

In a paper,* presented before the Northeastern Section of the American Chemical Society in Boston in January, 1903, I showed that when silica, alumina and lime, in proportions which would correspond to those in a Portland cement, and in which the iron was replaced by a molecular equivalent of alumina and the other bases by lime, were heated to a high temperature, 1,650° C., a product was obtained possessing, as was shown by the Newberrys, all the attributes of Portland cement, and which in thin sections under the microscope showed no evidence of the presence of aluminates, but was a homogeneous material which plainly consisted of a solution of the aluminate, 15%, in 85% of the tricalcic silicate. The conception of a solid solution, which we owe to the distinguished Dutch chemist, Van't Hoff, enables us to explain in this way at once the constitution and structure of Portland cement.

Objections have been raised to Newberrys' theory that dicalcic aluminate and tricalcic silicate were the constituents of Portland cement, because insufficient lime is present in any individual clinker to satisfy the amount required by these two compounds, but it has been shown by me that the aluminate $2\,Al_2O_3\,3\,CaO$ and the monocalcic aluminate $Al_2O_3\,CaO$ are equally soluble in tricalcic silicate, and may be substituted for the dicalcic form. If the amount of lime is still further reduced, dicalcic silicate forms and the cement is unstable.

The iron and the remaining bases add nothing to the hydraulic properties of the clinker, and may be very variable in amount. They merely form a flux or magma of low melting point out of which the tricalcic silicate with the aluminate in solution crystalizes and which can vary in basicity without essentially altering the character of the cement.

This subject can hardly be gone into in this place at greater length and its thorough exposition will appear later in "Cement." It is sufficient to say that while it had been previously suggested that Portland cement consists of a solid solution the proof of this has been first advanced in America. American chemists are, therefore, not only doing their share in making contributions to a higher knowledge of cement, but are well in advance of the times.

That there are no journals devoting themselves to the literature of the constitution of Portland cement is hardly true.

"Cement and Engineering News," of Chicago, reprinted in its pages the Newberrys' paper, which has been referred to, at the time of its appearance, and also issued it in separate form, The same journal has recently published a further communication of great interest on the same subject by S. B. Newberry, while "Cement" is now publishing an article of purely scientific aspect on the "Constitution of Portland Cement." The "Journal of the American Chemical Society" has also published two papers by Campbell of the University of Michigan; the second of which is of interest as showing the effect of fine grinding of the raw materials. It would seem, therefore, that a mechanical engineer like Mr. Orton should have made himself better acquainted with the literature of the chemistry of cement before making the assertion that he has put forward.

I should be very glad to show Mr. Orton, or others interested in the subject, thin sections of the synthetic Portland cement which has been made in my laboratory which I am sure will convince him that American chemists are not at all behind the times in a higher knowledge of the constitution of cement.

Mr. Orton's assertion that the recourse must be had to Germany for a reliable Portland cement hardly needs consideration in an engineering journal as engineers are well convinced by this time of the absurdity of any such claim.

With great regret that the paper of Mr. Orton had not been brought to the attention of cement chemists and manufacturers before it appeared in your pages, I am,

Yours truly, Clifford Richardson.

West Ave. and 6th St., Long Island City, N. Y., Jan. 18, 1904.

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*Abstract "Thonindustrie Zeitung, 1903, No. 58,942, May 16, Portland-Zement als feste Lösung."

American Portland Cement and American Contributions to its Chemistry.

Sir: In your valuable journal, under date of Jan. 14, 1904, under the head of "Opportunities for the Chemical Engineer in the Ceramic Industry" (extracts from a paper published in the "Bulletin of the University of Wisconsin," No. 83), by Edward Orton, Jr., E. M., I find certain remarkable statements as to the Portland cement industry in this country.

As one of the pioneers in this business, having devoted nearly a life-time to the introduction and manufacture of American Portland cement; to the study of the various properties of the materials entering into the manufacture of, and the many new uses for the same, I feel like exclaiming, like that good old poet Burns, "O wad some Power the giftie gie us, to see oursels as ithers see us!"

While giving the American cement industry proper credit for its wonderful manufacturing genius, for the marvelous output and economy of the mills, Professor Orton takes up the work of the American cement chemist and the technical management of the American cement mills, and criticises both severely. Certainly the first thing in chemistry is accuracy, and certainly the first test of an article such as is written, criticising the industry in its technical management, as Professor Orton has done, involves absolute accuracy on his part in any statement of views. Tested by this measure, it is only necessary to state that he sums up the American output as follows:

> We produce six or eight millions of barrels a year, but we have produced scarcely a single investigator who has contributed anything noteworthy to the knowledge of cements.

The Geological Survey of the United States, in its annual report for 1902, gives the output of Portland cement for the year 1901 as 12,711,000 barrels, and for 1902 as 17,730,000 barrels, and the year 1903, which has just closed, will certainly show very nearly 20,000,000 barrels. Were these figures the only ones in existence they might be the subject of doubt, but the "Mineral Industry of the United States," published by the "Engineering and Mining Journal," an equally official publication, gives the same figures for the year 1901, and for 1902 gives 16,535,000 barrels, as a result of its own independent investigation. It may be well asked by Professor Orton, in view of figures such as these, "what, then, are we to conclude from this hasty view of the American Portland cement industry?" If technical accuracy, such as is recommended; synthetic chemical processes, such as are suggested; scientific criticism, such as is stated to be essential, produce such inaccuracies as above mentioned, possibly in an industry where the difference of one-half per cent. in lime may mean success or failure, this sort of knowledge might best be excluded.

Viewing the subject, however, as it is viewed by Professor Orton, can it be for a moment admitted that the American Portland cement industry is one single step behind the cement industry of the world? To this there seems to be but a single answer, Most unquestionably, No! In mechanical equipment, in scientific attainment, in unlimited production, the American Portland cement industry is second to none in the world. To the writer's own knowledge it is considered the proper place for cement experts of the world to come for their post-graduate course. In a single month, at Allentown, Pa., there were registered experts from Spain, from Cuba, from Germany, from China and from Japan. All of them were attracted by the wonderful development of this great industry in its native home in this country of ours.

As a writer upon cement topics, and in that respect in thorough touch with the literature of the world; as a manufacturer of cement, and in that respect familiar with all its details, both chemical, mechanical and practical; as an importer and shipper of cement, and in this respect familiar with the markets of the world and the character of the various cements of the world, I want to most unqualifiedly deny the following statement by Professor Orton:

> The American cement industry needs two things at present before it can expect to win the choice trade of the world. We may undersell Germany, if quality is not absolutely imperative. But where reliability is the first essential the Germans have us on the hip.

These words cannot be used towards the American Portland cement industry without awakening a universal response and denial.

In the markets of the world, Japan, Cuba, South Africa, England and in this country, wherever American cement has met the foreign cement, it has proved more than its equal in all of the qualities that go to make up the sum total of the value of a good building material, and this fact is not denied by those on the other side, who with broad minds have viewed the development of the American Portland cement industry and the quality of its output.

Certainly the great cement plants of this country, with their wonderful mechanical equipment for handling the raw material, grinding and burning it, and ultimately producing the finished product, are unsurpassed in mechanical point of view by any in the world, and their chemical laboratories, testing-rooms and other appliances of a scientific character are, so far as the works in the East are concerned, and to my knowledge in many of the works in the West, fully equal to any of those I have seen in my visits to cement works in Europe, in point of equipment, character of apparatus and in the intelligence and ability of their technical managers.

It is impossible to believe that the millions of dollars that have poured into this industry, and the millions of dollars this country has paid for the use of American Portland cement in works involving hundreds and thousands of millions of dollars, that this confidence would have been reposed in an article so unworthy of scientific belief and scientific confidence, as it is claimed to be in Professor Orton's article.

So much for the production of the material and the engineering and scientific confidence accorded to it.

As to the lack of American literature on the scientific side of Portland cement making, it is only necessary to say that to any one who is in the current of American or foreign literature, and who "reads as he runs" the literature of this country on both the mechanical and scientific side of cement making, is the equal of any of the literature of the world.

Certainly, from the chemical side, the papers of Newberry, Richardson and Campbell are quoted in all the journals, English, French and German. The papers of Eckel on "Slag Cement," "Classification of the Crystalline Cements," "Portland Cement Manufacture," are well known wherever cement is spoken of. Meade's paper on the "Chemical Analysis of Portland Cement, Its Possibilities and Limitations," Green's paper on the "Manufacture of Cement," and Whitfield and Lewis' writings, are also well known.

Lathbury's article on "The Rotary Kiln in Portland Cement Manufacture," Spackman on the "Production of Marl Cement," Green's "Portland Cement Industry of the World," Hillebrand's paper on the "Work of the Sub-Committee of the Society of Chemical Industry on the Analysis of Materials for the Portland Cement Industry," Maclay's paper on "Hot Tests for Determining the Change of Volume in Portland Cement," Lundteigen's paper on the same subject, Newberry on "Fuel Consumption in Cement Burning," etc., etc., may also be mentioned. These are but a few that come quickly to hand from the large collection of articles, clippings and books which are in the writer's possession.

In conclusion, as to the deficiency of our literature claimed by Professor Orton, the writer can only say that in addition to the above articles he has been for the past few years indexing and filing everything on the subject of cement, and in that short period has gathered almost 1,000 long, complete and full scientific articles of American authority, exclusive of short newspaper clippings.

For a deficient trade and scientific literature, this seems to be doing pretty well, though possibly it covers some matters outside of the immediate field of that interesting chemical subject known as "silicates."

From the few statements above gathered, it can readily be seen how the American cement manufacturers, chemists and experts must either have fooled themselves, or have been successful in fooling the world at large. It is just such writings as the one that is criticised here, that brings the men who have spent the heat and the labor of the day, to the fore, and to a realization of the importance, seriousness and value of their work. Side-lights such as Professor Orton throws upon our industry, certainly are beneficial in bringing us to a realization of what we have achieved and what we are really doing, and to the world-wide importance of our work.

Robert W. Lesley, Associate Am. Soc. C. E.
22 S. Fifteenth St., Philadelphia, Pa., Jan. 27, 1904.

Concerning the Failures of Reinforced Concrete.

Sir: It seems to the writer that the position of Mr. Julius Kahn, in your issue of Jan. 21, on the necessity of some form of diagonal tie in steel concrete beams, and also on the advisability of using shapes of steel that do not rely solely on the adhesion of the concrete, is well taken. It would seem that both these advantages could be combined with that of flexibility of design and construction, by using stirrups in connection with some of the deformed rods now on the market. The stirrups to be made of either plain or deformed rods, bent U-shape to fit snugly the small part of the deformed rod and applied so as to bear against the enlarged part of the rod and extend out at an angle of 45° therewith.

The writer offers this suggestion, hoping it may prove a useful contribution to the rapidly increasing number of designs for re-enforced concrete.

Yours truly,
R. M. Walker.

Atlanta, Ga., Jan. 26, 1904.

Sir: Mr. Kahn's paper in your issue of Jan. 21 touches on several features of interest to an engineer in the contracting field. I would like to hear from him further regarding the beam illustrated. If we assume the concrete to be a 1-3-6 mixture aged six months, then by Mr. Kimball's tests, as quoted by Mr. Thacher in his writings, this concrete would have an ultimate crushing strength of about 3,100 lbs. per sq. in., and a modulus of elasticity of 1,860,000. For such concrete Mr. Thacher has shown that 1 sq. in. of steel for the beam given would be sufficient, as against 1.69 sq. ins. specified by Mr. Kahn, and that then the maximum moment would be about 90,000 ft. lbs., corresponding to a uniformly distributed ultimate load of 72,000 lbs. and a maximum end shear of 36,000 lbs or 300 lbs. per sq. in., whereas for the above grade of concrete, Mr. Kahn specifies steel sufficient to require a crushing strength of far from 3,900 lbs. per sq. in.

If 300 lbs. per sq. in. is thought to be an excessive ultimate shearing strength, or if the safe limit is to be 50 lbs. or less, then a modification of the section of the beam making it shallower and thicker would be in order.

Yours truly,
E. Landor.

Canton, Ohio, Jan. 23, 1904.

Sir: Referring to the article on "Failures of Reinforced Concrete," by Mr. Julius Kahn, in your issue of Jan. 21, 1904, the failures or cracks in the beams which Mr. Kahn says are due to shear on the concrete, or lack of vertical reinforcement, are characteristic cracks commonly found in such beams when the span is short as compared with the depth. In long and shallow beams they are not usually found. When such cracks are formed near the ends of the beams they are invariably followed by the metal reinforcement pulling out at the ends. This will be found true in spite of all care that may be taken to insure a bond between the metal and the concrete, and without bringing any apparent strain on the metal. The writer is prepared to show that such cracks are due to a slipping of the metal in the concrete, and are not due to a shear. The term shear is frequently misapplied by writers, and should be rigidly confined to a sliding between two contiguous surfaces.

In making some tests on reinforced concrete beams about a year ago, in which the characteristic cracks appeared, accompanied by a pulling out of the metal at the ends, it was suggested that if the bars could in some way be anchored at the ends they would not pull out. This seemed rational, for if the bars could not slip at the ends they would certainly not start to slip anywhere else. The subsequent tests made on such beams proved the reasoning to be correct, inasmuch as the characteristic cracks disappeared and failure invariably occurred in the tension flange, where it should occur.

In order to accomplish the desired result, all that was necessary was to introduce a transverse rod at each end of the beam and hook the longitudinal rods over these. All this was done with plain material bent cold in the field.

This method of reinforcing has been in use in this office ever since, and has been successfully employed in building concrete girders for flat-top culverts in spans up to 20 ft. It has also been used in designing a retaining wall 26 ft. high. In fact, it is used in all cases where no dependence can be placed upon the adhesion of the mortar to the metal.

Yours truly,
J. W. Schaub, M. Am. Soc. C. E.,
Consulting Engineer.

1630 Monadnock Block, Chicago, Ill.

Sir: While Mr. Kahn in his article of Jan. 21 has taken great pains to explain from the theoretical side of the subject some of the causes of recent failures of reinforced concrete, he has left the practical side of the question untouched. I agree with Mr. Kahn relative to the causes he ascribes to the failure, but there are some causes from the practical side to be considered.

First, lack of judgment in the man in charge of the work in permitting it to be done in freezing weather; second, bad material and mixing; third, lack of thought in drawing up the specifications.

In giving my opinion, I want to state that I am not an engineer, but I have worked on a number of the large buildings where reinforced concrete was used, which have been erected in New York, within the past six years. Any person can prove to his own satisfaction the result of frost penetrating concrete by taking a piece of the same and submerging it in water and seeing how long it takes to crumble.

On two of the buildings the segmental arch was used, with the ordinary wire cloth center, which had steel rods ½-in. in diameter running transversely 8 ins. apart and ¾-in. bars horizontally. The arches were constructed in a temperature of about 26° F., and the material was mixed very wet, with the result that a large percentage of the cement was wasted by the leakage of the water through the arch, and the concrete was frozen before it had time to set. The consequence was that the concrete expanded; the frost destroyed the bond of the cement, and when a thaw came, the concrete contracted, and the cement having lost its strength, the mass was nothing more than a pile of cinders.

On another building, a flat arch was used which was reinforced with two steel bars running horizontally with cross bars 16 ins. to 18 ins. apart. The arch was constructed in a temperature below 26° F., and the centers were left in for more than a week. When they were removed, sections of the concrete fell through that part of the floor which was constructed during the freezing weather.

On another building, small broken stone was used in place of cinders; the arches were reinforced the same as

those last described; the work was done in summer, and the centers were withdrawn after three days. Some sections of this concrete also fell. The reason for this was undoubtedly due to the theory advanced by Mr. Kahn, the reinforcement was insufficient to carry its weight.

While I agree with the inspectors and the foreman that concrete mixed very wet is better in walls where the leakage can be stopped or confined to the forms (as it saves the labor of tamping and insures against destroying the bond of the layer of concrete immediately underneath), I do not agree with them in having the concrete mixed so wet in the case of reinforced arches where the water leaks through the centers as through a sieve.

It is wiser to mix the concrete just wet enough to keep it from running, and pay for the extra labor of tamping, as it insures against an unnecessary loss of the tensile strength of the cement, which is the chief ingredient of the concrete, thus helping to prevent the danger of collapse.

Why architects and engineers specify the flat arch (which they know is more liable to sag and is defective in shear) in preference to the segmental arch (which they also know will shear to the I-beams, thus transmitting the weight horisontally to the walls and columns), is a mystery to me.

Yours respectfully,

William Martin.

1009 East 42d St., Brooklyn, N. Y., Feb. 3, 1904.

Sir: The writer has read the article on "Some of the Causes of Recent Failures of Reinforced Concrete" published in your issue of Jan. 21, and was much struck with the author's generous philanthropy, which does not allow him to keep to himself the knowledge of a specific certain to cure all our ills.

Mr. Kahn seems to consider himself as the original discoverer of the shearing stresses in reinforced concrete beams, which he seems to have made about a year ago, more or less. The writer protests that this is hardly fair, when he knows of a number of prominent firms in this country and abroad who have been taking care of these stresses in their own work for the last eight years. The writer's company had published a catalogue in which these matters were discussed, and formula given for indicating when provision against shear was to be supplied, some months before Mr. Kahn had "found himself."

Concerning some ridiculous expanded metal floor construction referred to in this article, the writer wishes to state that there are a large number of expanded metal companies throughout this country, all of whom design their own work, and they have nothing to do with each other. Some of them, of which the writer's company is one, are not on good terms with the others, and have no knowledge of what they are doing. That some of them have not employed competent engineers, has been shown in a number of cases, and the writer objects to being placed in the same class with them.

With what Mr. Kahn says about the necessity of competent advice in the design and execution of steel-concrete structures, the writer fully concurs. But he does not follow him to the extent of saying that Mr. Kahn, for the one, and his bar for the other, are essential to success.

So far as Mr. Kahn's theories are concerned, he makes the mistake of assuming that all steel-concrete beams fail by shear. It depends upon the ratio of height to length, and the character of the loading, just as in any other kind of beam.

He makes a further mistake in his discussion of the shear in the beam illustrated in his Fig. 1. The safe carrying capacity of such a beam is not 25,000 lbs. of uniformly distributed load, as he states, but just one-half of this, 12,500 lbs. The former would be the safe carrying capacity for a beam of the depth shown, 12 ins. wide, but his beam is only 6 ins. wide. The above values are based on a factor of safety of 4, and the ultimate carrying capacity, then, should be 50,000 lbs. instead of 100,000 lbs., as he states. He also says that, assuming a safe shearing stress of 50 lbs. per sq. in., the cross-sectional area being 120 sq. ins., "that the beam would only carry a safe load of 6,000 lbs. in shear," and he makes a comparison between this and the 25,000 lbs. safe load against bending, giving an apparent discrepancy of 1:4 as between the strength against shear and the strength against rupture by bending. The 6,000 lbs. above mentioned is not the safe load on the beam against shear, but the reaction at one end under such safe load, the load itself being twice this, or 12,000 lbs., and the comparison should be made between this 12,000 lbs. and the 12,500 lbs., the safe load for the bending moment. In other words, the safe load that he gives against shear is only one-half what it should be, and the safe load against bending is twice what it should be, and the values, instead of showing a ratio of only 1:4, are in fact practically equivalent. He says, later on, that he has not obtained shearing results on concrete larger than 250 lbs. to 300 lbs. per sq. in., which indicates that he has taken a factor of safety of 5 on shearing, and 4 on bending. If we compare the ultimate moment of resistance of such a beam with the ultimate shearing strength as indicated by these values, we will have a maximum ultimate shear at one support of 250 × 120 = 30,000 lbs., giving a load carrying capacity of 60,000 lbs. on the beam, as against the maximum load possible to develop by the moment of resistance of 50,000 lbs. By the above discussion, therefore, using Mr. Kahn's own figures and simply correcting them where they are manifestly in error, it is evident that the statement made in the last sentence of the paragraph, to the effect that "it is evident that a beam thus reinforced is exceedingly weak in shear," is not warranted. Of course such a discussion as the above has no pertinency whatever to those who have any knowledge of the maximum shearing stresses in reinforced concrete beams, and the fact that Mr. Kahn should take the trouble to develop, however carelessly, the shearing stresses in the vertical section, merely indicates that he has no real knowledge of how or where the maximum shearing stresses in such beams are developed.

So far as his bar is concerned, if each wing were separated from the rectangular middle portion, the latter allowed to pass horizontally to the end of the beam, and the side rods have their ends bent up, one having the bends at the quarter point and the other between this and the end, we would have the equivalent of the Kahn bar for nearly all conditions in practice, wherein the maximum load is seldom a concentrated load. In fact, the use of separate rods in this way, which is the common practice, besides being cheaper, has the advantage of permitting the separation of the parts so as to have each part entirely surrounded by concrete. Besides negative advantages, his bar has certain positive disadvantages. For example, in the usual case of maximum load being approximately uniformly distributed, the necessity for shearing provision occurs only adjacent to the supports. For this case, the Kahn bar would then have a considerable length in the middle wherein dependence would have to be placed upon the adhesion, which he admits is not wise. To obviate this by cutting the wings and bending them up from end to end involves a heavy loss in metal. Furthermore, the bar must be symmetrical around the middle portion, and hence must be prepared in advance for a particular place, and cannot be economically used in other places.

In floor construction the usual practice of good companies is to let half the bars run through horizontally and half bend up from about the quarter point, and hook over the supporting girder beams. Taking a flat slab design, the writer's tables give for 12-ft. span, using cinder concrete, a 6-in. slab, with 0.3-sq. in. of metal per foot of width, as capable of carrying 615 lbs. per sq. ft. ultimate load uniformly distributed. The writer never heard of such a construction failing by shear, though the total load is twice what it would be if the beam were simply supported at the ends. The writer has made numerous tests himself on such work and knows that this is true.

In some eighteen beams recently tested at the Rose Polytechnic Institute by Professor Malverd A. Howe, ranging in size from 5 ins. deep by 12 ins. wide and 10-ft. span, to 21 ins. deep by 12 ins. wide and 18-ft. span, three failed by shear at the ends within about 15% of the ultimate strength in bending, the others giving beam breaks. With regard to these three, the tests on compression blocks indicate that most of this discrepancy was due to the concrete not having the theoretical crushing strength assumed. Professor Howe will no doubt wish to write up his own investigation, and the writer does not wish to anticipate him by giving the details at this time. Suffice it to say that they will show that there are several roads to Rome.

Very truly yours, A. L. Johnson, M. Am. Soc. C.E.

St. Louis, Mo., Jan. 23, 1904.

Sir: In the issue of Engineering News for Jan. 21, 1904, there appeared an article by Mr. Julius Kahn, Assoc. M. Am. Soc. C. E., entitled "Some of the Causes of Recent Failures of Reinforced Concrete Construction."

Mr. Kahn confined his remarks principally to the proper distribution of the steel reinforcement, and stated, in effect, that all other matters were of secondary importance. From his point of view perhaps this is correct, but there are at the present time in use quite a number of "systems" of reinforcement, different shapes of rods, sheets of metal, wire cloth, etc., all of which, when properly used and tested to destruction, have given excellent results. Many of these systems have been in use, and the materials been on the market for a number of years, and if the systems themselves were at fault the number of recorded failures would have been many times increased.

That the steel, in some systems of reinforcement, is not economically distributed can scarcely be denied, but the fact that beams and slabs made under any particular method (size, shape, etc., being uniform) show a quite uniform breaking strength, indicates the capacity of that system. The fact that some of the beams, which are made and tested alike, break at a much lower stress than others, should not detract from the merits of the system, because its capacity has been indicated by the highest test of the series. The beams breaking at the lower stress simply indicate that in some respects they were inferior to others of the series, or in other words, defective.

If there is any great range of value for this test breaking stress it should be the duty of the person conducting the tests to ascertain the cause, in order to guard against a repetition of the same. When this is done the results are not always published, and we sometimes see tables of breaking tests, on beams and slabs of reinforced concrete, varying by 50%, more or less, with no explanation of the variation.

In beams with the same amount and disposition of metal, and tested in the same manner, the cause of this variation in strength must be looked for in the composition of the concrete, the manner of placing, or the manner of removal of the forms. The beams made for test are usually made with greater care than can be exercised in the hurry of contract work, and the conditions as regards cleanliness are much better; therefore, any defects which we may find in the test beam are likely to be magnified many times, in actual construction.

While it is desirable to have the steel reinforcement placed as economically as possible, still, if a structure is designed to be built in accordance with any of the standard systems of reinforcement, using as a basis the results of full size tests for strength, with a proper factor of safety, the structure will be safe, provided all the work of construction is carefully and properly done. Notwithstanding this, we sometimes hear of a structure (or part of a structure) so designed which fails. Evidently, then, the last condition has not been fulfilled.

The writer believes that most of the failures of reinforced concrete can be traced to improper and careless handling of the materials, forms, etc., and while he has not had an opportunity to examine any of the larger failures, he has watched with considerable interest the process of construction in different places, and on works of different character, in which both concrete and steel are used, and has found it an excellent way to learn how not to build reinforced concrete.

It is undoubtedly true, that the majority of busy engineers and architects have comparatively little time to spend on the work being built according to their design. The contractor also is generally more interested in securing other contracts and in seeing that they are conducted expeditiously than in the actual details of construction. This is to be regretted as it leaves the work as a rule in the care of foremen and inspectors, who generally have no (or a very imperfect) idea of design. There seems to be a very prevalent idea among these men that the steel is intended to take all the stress, of every kind, and that the concrete is used simply to level up, or give form to the structure.

In support of this statement the writer would call attention to some of the different methods in vogue in building reinforced concrete as witnessed by him.

In examining some floor construction, of about 15 ft. span, not long ago, the writer noticed a floor panel which was about 1½ or 2 ins. lower on the top than the others, and this was explained by the foreman, who stated that there was still that much concrete to be placed thereon. It had been left in that shape at night, owing to a combination of circumstances which it is not necessary to relate, and at the time examined the concrete was quite hard. The composition of the concrete seemed quite rich, and owing to the ramming and water flushing to the top the surface, while irregular, was smooth in a way resembling vitrified fire clay. This panel was subsequently leveled up to the elevation of the others. From the writer's observation the adhesion of concrete to other that has set is a very uncertain quantity, there always being a well defined line of cleavage and the concrete being easily separated along that line. In the writer's opinion the effective depth of this panel was reduced by about the depth of the top layer of concrete, the latter giving little if any more strength than a plank laid on top of a beam.

In another case of floor construction the forms had all been constructed, the bottom layer of concrete had been spread over the form, and a laborer was in the act of placing the sheets of metal. The work adjacent had been completed and was set hard. The sheets of metal were slightly too long for the space they were to occupy, and instead of cutting them off or bending the end up, the ends were forced down to about the proper elevation and the whole sheet then forced down by walking on the top and striking with a rammer. This caused the sheet to buckle badly and in places it was forced down through the concrete clear to the form, while in others it stood about 2 ins. from the forms. As the placing of the concrete and the ramming progressed these high points would be forced down, and the metal would rise at other points. The metal and adjacent concrete were therefore in a state of agitation, which coupled with the fact that the concrete was rather too dry, the cement quick setting, and the time consumed in placing about one and one-half hours, makes it almost certain that the bond between the concrete and steel was imperfect, with the probability of voids also. Evidently the loading of such a floor would produce a tendency for the metal to straighten out under the tension, thereby separating the concrete below from that above the metal. That it has not so separated in this case is probably due to the fact that the span was short. The loading, also, at this point would probably not be as great as other portions of the floor.

In another case in which the forms, rods and sheets of metal were all in place before the placing of the concrete began, the concrete was dumped on top and forced

through the meshes of the metal by raising the latter with hooks. The concrete was dumped from wheelbarrows rolled on plank runways, which, together with the men ramming, occupied so much of the space that the raising of the metal and forcing of the concrete through the same was not properly or uniformly done. The foreman stated that he "Did not care if it was a little rough on the bottom as it had to be plastered." From his point of view the results must have been excellent, as when the forms were removed the bottom was certainly rough enough, and there were quite a number of places where the metal showed, varying in size from a few square inches to a matter of square feet.

The amount of exposed metal in the underside of some reinforced concrete floors would probably prove a surprise to their designers if they would examine the floors immediately after the removal of the forms.

It would seem to be self-evident to even the most ignorant mechanic that the forms should be removed with as great care and as little damage to the concrete as possible. Nevertheless, the writer recalls a case where some forms for the underside of a floor, made of matched boards and laid in full lengths, had swollen so much and adhered to the concrete so badly that their removal was rather difficult, and a laborer, with staylaths erected to protect him from the falling boards, was engaged in showering blows of a twelve-pound hammer against the underside of the floor in order to jar the boards loose.

How many blows of a similar hammer would it take to injure a soft sandstone slab 12 ft. square and 8 ins. thick, supported at all four sides?

The writer not long ago examined a structure in which small rods placed vertically were used as reinforcement for a vertical wall designed to act as a beam. The concrete had been placed for a height of about 5 ft., and at that point work had been suspended, apparently for some time, as the concrete was quite hard. The rods were held in a vertical position at a point about 3 ft. above the concrete by wires and blocks of wood, while at the top five to eight feet of rod was free to swing. A slight touch was sufficient to set these rods swaying, which they continued to do for some time. On examining the concrete where the rod entered, there was found to be an open space around the rod in which a knife blade could be run a couple of inches. From the angle through which the rod moved it was estimated that there was no adhesion between the concrete and rods for at least six inches below the surface.

It is usually assumed that the adhesion of concrete to steel is about 50 lbs. per sq. in. Assuming that the concrete in this case was laid in 9-in. layers, with the rods constantly swaying from the ramming, etc., what should be allowed for the adhesion of the concrete? The rods were of a deformed type which in this case was certainly a great advantage.

In the construction of some sidewalk vaults, built with concrete reinforced with expanded metal, between steel beams, it was noticed that the expanded metal was placed with the long diagonal of the mesh parallel with the steel beams, apparently only because the sheets cut up more economically. What is the comparative strength of the metal placed in this way?

That such practices in the use (or abuse) of materials in reinforced concrete construction are not common is to be hoped, and the writer believes such to be the case. Nevertheless, such cases do exist, for the above are not hypothetical cases, but observed facts, and many other cases of similar nature might be enumerated. Sufficient, however, has been said to show that the design of a structure in reinforced concrete need not be the only cause of its failure, and there are undoubtedly some such structures now standing simply because their weakest part has a factor of safety (due to construction) of 1 +.

In the writer's opinion this condition of affairs is caused by lack of sufficient and efficient inspection. It is seldom the case on any piece of construction that there is a sufficient number of competent inspectors, and this is particularly the case in building construction. The inspectors, also, frequently have so little authority as to make them little more than figureheads, or "spotters," on the work.

Such a condition of affairs is entirely wrong. From the writer's experience he has found that by far the greater number of contractors desire, and endeavor, to produce work perfectly safe, and the character of which cannot be questioned.

The source of danger lies in the laborers and foremen employed upon the work. The former desire only to make as many days' time, at as high a rate of pay as possible, while the latter knows that his value to his employer, and consequently the amount of his salary, depend principally upon the quantity of work done in a certain time. Both work on the assumption that anything which passes inspection is "good enough."

It is in correcting this impression, in pointing out defective work and insisting upon its removal and correction (preventing rather than correcting, wherever possible), that an inspector is of greatest value. He should be a man of experience in the kind of work of which he has charge, and should be given sufficient support and authority to enable him to see that all work is well and thoroughly done and in accordance with the specifications.

While it is probably true that structures of reinforced concrete have failed from improper designing, it is undoubtedly equally true that they have failed from improper construction.

In either case the engineer or designer is equally responsible, and the blame should not be laid upon the contractor, who more than likely desired to turn out a first-class piece of work. Very truly yours,

E. G. Haines, Assoc. M. Am. Soc. C. E., Resident Engineer, Division No. 1, Atlantic Ave. Improvement.
5 Hanson Place, Brooklyn, N. Y., Jan. 27, 1904.

(Proofs of the above letters were submitted to Mr. Kahn, and we append his reply as follows:

Sir: Replying first to Mr. Johnson:

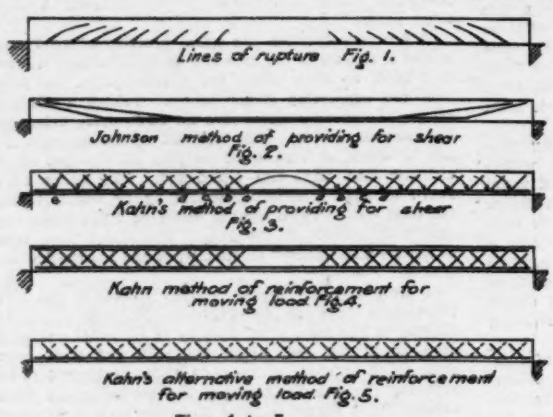
As for the writer's considering himself "the original discoverer of shearing stresses," the statement is absurd. As a matter of fact, reference was made by the writer in an article in Engineering News of Oct. 15, to Capt. John Stephen Sewell's excellent paper in the Engineering News of Jan. 29, 1903, in which Capt. Sewell so convincingly called attention to the matter of vertical reinforcement and the close relationship between theory and actual tests in concrete-steel. Although Capt. Sewell does not lay claim to the discovery of the need of such vertical reinforcement, he is certainly entitled to the credit for having called first attention to this in an American publication.

Nor is the writer unaware of the fact that such shearing stresses have been provided for by competent engineers heretofore. In fact, he said, in his recent article, that such engineers have rarely had failures in their structures.

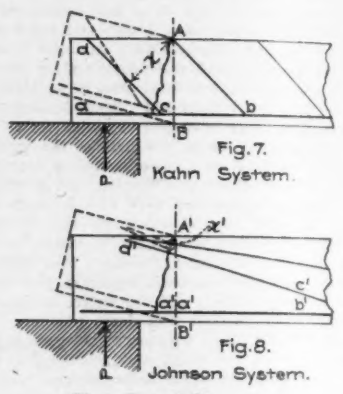
As for the Kahn bar being essential to success, not a single mention of it was made in the article in question. But since Mr. Johnson attacks it, let us consider his objections.

First, as to his statement that "if each wing were separated from the rectangular," etc. Authorities agree that even plate girders should be stiffened at distances not farther apart than the depth of the girder, and that the stiffening should preferably have an inclination of 45° to the vertical. (See Theory of Framed Structures by Prof. J. B. Johnson.) And yet, in a concrete beam where the safe shearing capacity of the material is only about 50 or 60 pounds per sq. in., Mr. A. L. Johnson thinks that the turning up of the bars at the quarter point and midway between there and the ends (Fig. 2), not even providing the generally used stirrups, is quite sufficient. It is well known that the efficiency of the shear reinforcement becomes greater as it approaches the vertical, and yet in the case of a beam, say 14 ins. deep, 24 ft. span, Mr. Johnson believes that the reinforcement set at an angle of 70° or 80° with the vertical, is quite enough. Does this seem reasonable? Furthermore, if maximum shear does exist primarily at the ends, is not the Kahn bar (Fig. 3) with its shear members at an angle of 45°, and at distances apart equal to the depth of the beam, more efficient than Mr. Johnson's set at 70° or 80° with the vertical?

Experiments made by the writer, as well as by others, have indicated that the best manner of regarding the stresses set up within a concrete beam, is to consider them as internal arches of stress, as shown in Fig. 1, each arch having its horizontal thrust taken up either by the adhesion of the concrete to the horizontal bar, or by positive abutments provided by the Kahn bar, which transmit the stress into the horizontal bar. To satisfy himself conclusively on this point, the writer has taken blocks of hard material formed as the voussoirs of a flat arch, and set them between the prongs of his reinforcement bar. These, although not a particle of cement had been used between the various blocks, carried large weight, and had it not been for the deflection due to poorly fitting joints, would probably have been as strong as if the beam had been a solid compact mass. The prongs formed positive abutments for each voussoir, or arch of stress;


Lines of rupture Fig. 1.
Johnson method of providing for shear Fig. 2.
Kahn's method of providing for shear Fig. 3.
Kahn method of reinforcement for moving load Fig. 4.
Kahn's alternative method of reinforcement for moving load Fig. 5.

Figs. 1 to 5.



Figs. 7 and 8.

TESTS OF CONCRETE-STEEL BEAMS MADE AT THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

No. of beam.	Age, in days.	Number, size and kind of bars—Near bottom—Side No.	Side of sq.ins.	P or T.*	Near top—Side No.	Side of sq.ins.	P or T.*	Manner of load'g; time of fracture.	Weight in lbs. Of beam.	Breaking load.†	Max. bending moment at fracture, in.-lbs.	Remarks.
1	40	1	...	..	..	...	..	Center.	1,198	1,302	62,733	Vertical break near center span.
2	40	1	¼	T	..	...	..	Center.	1,200	1,300	62,700	Vertical break at middle. Bar drew down to ⅛-in. and broke.
3	30	1	⅛	T	..	...	..	At two points	1,205	10,095	241,973	Vertical break at one point of application of load. Bar drew down to ⅛-in. and broke.
4	38	1	⅜	T	..	...	..	Center.	1,160	13,680	470,580	Concrete crush on top.
5	50	1	⅜	T	..	...	..	"	1,290	14,710	506,715	
6	50	1	1	T	..	...	..	"	1,204	15,796	541,134	
7	41	1	1¼	T	..	...	..	"	1,195	12,805	442,283	
8	41	2	1	T	..	...	..	"	1,240	18,760	639,540	
9	42	2	1¼	T	..	...	..	"	1,274	23,105	783,486	
10	42	2	1¼	T	1	½	T	"	1,279	21,105	717,569	
11	45	2	1¼	T	2	¾	T	"	1,294	23,105	783,816	
12	30	2	1¼	T	..	...	..	At two points	1,282	24,200	553,553	
13	31	2	1¼	T	..	...	..	"	1,297	29,200	663,718	
14	30	2	1¼	T	2	¾	T	"	1,341	24,200	554,527	
15	53	1	1	P	..	...	..	"	1,292	15,250	356,818	
16	49	1	1	T	..	...	..	"	1,211	16,500	282,982	
17	43	2	¾	P	..	...	..	"	1,271	15,950	370,700	
18	40	2	¾	T	..	...	..	"	1,200	19,000	438,807	
19	35	4	½	P	..	...	..	"	1,261	17,500	378,095	Beam had crack 18 ins. from center; failed at this vertical crack
20	33	4	½	T	..	...	..	"	1,213	20,000	433,329	Failed by longitudinal shearing.
21	57	1	¾	P	..	...	..	"	1,213	12,500	295,015	Vertical break, point application load; rod pulled through; concrete not sufficiently packed in making.
22	54	2	⅜	T	..	...	..	"	1,248	22,250	510,092	
23	57	2	⅜	P	..	...	..	"	1,221	20,250	465,647	Beam cracked in handling, 22 ins. from center; failed from longitudinal shearing.
24	47	4	⅜	T	..	...	..	"	1,203	19,250	443,350	
25	50	4	⅜	P	..	...	..	"	1,192	15,250	355,168	
26	40	2	¾	T	..	...	..	"	1,215	24,250	553,548	
27	40	1	1	T	..	...	..	"	1,222	21,750	498,688	

*P—plain; T—twisted.
†Exclusive of weights of beam.

All of the fractures except Nos 1, 2, 3, 4, 19, 20 and 21 were of the following general character: They consisted of a longitudinal shearing break at, or a little above, the reinforcing bar, and, in addition, a break extending diagonally (often at an inclination of about 45°) upwards towards the center. The point where this diagonal break joined the top of the beam was sometimes at the point of application, and sometimes near it.

Note.—Beams Nos. 12, 13, 14, had U-shaped stirrups set so as to surround the bars.

and the greater the load, the greater the clinching action between the voussoirs on these prongs. Such a construction, then, forms a self-contained flat arch. Every voussoir has its abutment and every pair of abutments its tie rod. Each diagonal receives the summation of the vertical components of all arches of stress between it and the center of the beam at its bottom, and carries this load upward, spreading it over the arches extending between it and the main support of the beam. At the same time, it ties together these arches by crossing them practically at right angles, making them all act in unison. It is quite possible that some engineers may not agree with the writer, that reinforced concrete beams tend to resist stresses in the manner outlined above. Nevertheless, if records of actual tests to failure are studied carefully, it will be observed that almost always the break is of a nature corresponding to arches of stress, and the beam has failed because such arches found no abutment of sufficient strength to take their thrust, therefore failing by longitudinal shear along the bar, or vertically because means were not provided to transmit the vertical stress to the various arches and to cause them to act jointly.

FIG. 6.

Assuming then, that such arches of stress exist, as in Fig. 1, let us compare the writer's method of providing for these stresses with Mr. Johnson's proposed method. The accompanying diagrams are quite explanatory, and hardly need a special argument to show the superiority of one above the other. Undoubtedly the arrangement shown in Fig. 2 has some efficiency in shear, but the writer doubts its absolute and positive nature, and tests to destruction certainly give proof to his conviction. As for his own method of vertical reinforcement, Fig. 3, he has made so many tests with never so much as a hair of a shear crack, that he is warranted in positively asserting that shear is entirely provided for.

Referring to Figs. 7 and 8, notations marked thus, A B, refer to the writer's method of reinforcement, and those marked thus, A' B', to Mr. Johnson's method. Assume a given plane A B, the reaction P tends to revolve the end of the beam about the point A into the dotted position, and is resisted from doing so by the steel along the lines a b and c d in the writer's diagram, and a' b' and c' d' in Mr. Johnson's. Would Mr. Johnson explain why c d with the lever arm X should be especially less efficient than c' d' with the lever arm X'? If Mr. Johnson wished to stop a wheel from revolving, would he apply the resisting force along the periphery or somewhere near the hub? But perhaps Mr. Johnson will feel more at ease if we quote from the late Prof. J. B. Johnson regarding the distribution of shearing stress in the beam:

> It is proved in mechanics that wherever a shearing stress acts along any plane in an elastic body, there is another shearing stress of the same intensity at that point acting on another plane at right angles to the first. Also, that the effect of these two equal shearing stresses at right angles to each other, is to produce two direct stresses of the same intensity, also at right angles to each other and at 45° with the former planes, one of these direct stresses being tensile and the other compression.

According to this principle, which was originally outlined by Rankine, if we were to lay out the lines of principal tensile stress in the beam, we would find them to be curves approximating very closely a line of 45° with the vertical. The close relationship of actual failure lines with the lines of principal tensile stress was so clearly set forth in the article already mentioned by Capt. Sewell, that the writer can do no better than refer to the same.

In steel bridge construction, it is generally recognized that perhaps the most economical and efficient construction is the Pratt truss. Surely the efficiency of this arrangement of members does not suddenly cease when they are surrounded by a mass of concrete?

Regarding the 20-in. beam, cited in the recent article, the writer made a slight error in his statement that if the safe shearing capacity of concrete is 50 lbs. per sq. in., and if the area of shear is 120 sq. ins., the safe load on the beam would be 6,000 lbs. instead of 12,000 lbs. With 12,000 lbs. safe load, so far as shear is concerned, the beam would then be capable of carrying less than half of the capacity it has in bending, which is 25,000 lbs., provided the steel had been strained to its full value at the center.

If we regard Mr. Johnson's own published diagrams for the strength of steel concrete beams, we will find for a beam 12 ins. wide, 1½ sq. ins. of metal and whose depth is 19 ins., from the top to center of reinforcement, an ultimate moment of resistance of 1,600,000 in.-lbs. which would be equivalent to an ultimate distributed load of approximately 106,000 lbs., or a safe load of 26,500 lbs. Now, if a beam 12 ins. wide with 1½ sq. ins. of metal carries safely 26,500 lbs., a beam 6 ins. wide with ¾ sq. in. of metal should carry safely 13,250 lbs.

But Mr. Johnson, after his very lengthy discussion, would have us believe that this same beam with 1½ sq. ins. of metal is not a particle stronger. If Mr. Johnson had been consistent with the derivation of his own formulae he would at once have conceded that if two beams are taken into comparison the one with twice the quantity of steel of the other, the neutral axis must have approached necessarily the bottom of the beam and therefore the area of compression increased. Mr. Johnson neglects this matter entirely. Furthermore the writer disagrees with Mr. Johnson in the very location of his neutral axis, which determines his moment of resistance, and of course can not agree in the accuracy of his formulae.

If a construction made up of blocks of material or voussoirs, set between the prongs of the writer's bar without any adhesive material whatsoever, can be made to carry large weights, it is evident that not a very large part of such a beam can be in tension, and that arches of stress exist everywhere throughout the body of the beam, each finding its abutment in a set of prongs. It would seem more probable, therefore, that the neutral axis with such reinforcement is somewhere nearer the bottom of the beam than half or two-thirds of the way upwards.

The writer sees no objection of course to applying Mr. Johnson's formulae to beams reinforced with corrugated bars, as he has never known them to fail otherwise than in shear, but as for the writer's method of reinforcement, he has never known them to fail otherwise than in bending moment. In the case at hand he would not hesitate to load the beam to twice Mr. Johnson's determined amount.

As a matter of fact, a beam of this type has generally on top a concrete floor slab, which forms a very powerful flange in compression, and the beam need, therefore, not be made unnecessarily wide. The question of efficiency of reinforcement is whether the full tensile strength of the steel is transmitted to the body of the beam to balance this compression. If the stress coming into the horizontal bar is due to the horizontal components of inclined steel members attached directly to it, there is very little object in making the beam 12 ins. wide. In actual construction the writer would batter the size of his beam so as to make it possibly 4 ins. wide at the bottom and 8 ins. at the top, thereby increasing the compressive area of the concrete in the body of the beam. Since he depends so little on the adhesion between the concrete and the steel, and since, so far as strength alone is concerned, he might actually place his main longitudinal reinforcement outside of the concrete entirely, as is shown in test Figs. 9 and 10, he feels that only enough concrete is required in the lower part of his beam to thoroughly protect the steel. The beam should, therefore, be narrow at the bottom and batter towards the top. Such a construction is highly economical in the use of concrete as it places this material in a position where it is most efficient. So far as the safe loading is concerned for this beam, perhaps photographs of an actual test would be most conclusive, and the writer begs to refer to the test illustrated in Engineering News of Oct. 17, where two beams 20 ins. deep, each having 2 sq. ins. of steel im-

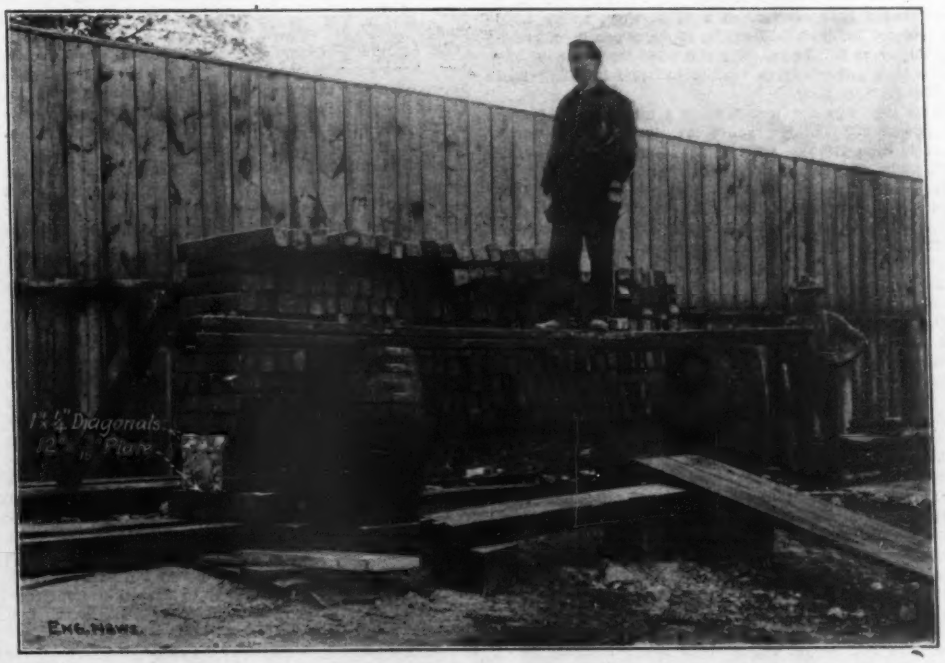


FIG. 9. LONGITUDINAL REINFORCEMENT ENTIRELY OUTSIDE OF BEAM. DIAGONALS ONLY EXTENDING INTO SAME. LOAD, 40,720 LBS. DEFLECTION, ¼-IN.

bedded therein, reinforced with the writer's method, carried 110,000 lbs. If on a 26-ft. span each beam with 2 sq. ins. of steel, carried 55,000 lbs., then in a 10-ft. span a beam with 1½ sq. ins. should carry approximately

$$\frac{55{,}000 \times 26 \times 1\frac{1}{2}}{10 \times 2} = 107{,}250 \text{ lbs.}$$

or 26,000 lbs. safely if a factor of safety of 4 is assumed. According to Mr. Johnson's formulae and mode of construction this result would have been impossible.

Further, while Mr. Johnson's arrangement might act effectively under certain methods of loading, does it seem

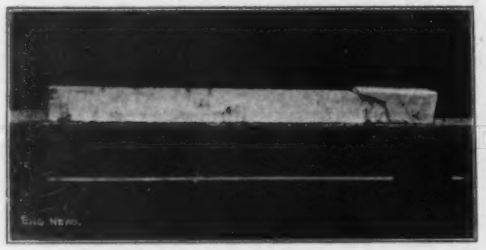


Fig. 11.

Fig. 12.

Fig. 13.

as though the metal had been placed in a reasonable manner where moving loads are taken into account, and where the stress diagram changes for every position of loading? As for the writer's method of taking care of such moving loads, he adopts either one of the two arrangements shown by Figs. 4 and 5. He sets his bar along the upper edge, as well as the lower edge, producing thereby virtually latticed girders, without any riveting whatsoever. Where a heavy reinforced slab must resist moving loads, and where an abundance of vertical reinforcement is not so necessary as in beams, the writer generally allows his bars to be sheared all in one direction, setting one set of bars so that all prongs point in one direction, and alternative bars so that the prongs point in the opposite direction. This gives him the same latticed effect without the top reinforcement bar, and at the same time obviates the necessity of cutting to special lengths, so objectionable to Mr. Johnson.

Regarding the matter of shear reinforcement for floor slabs, for which case Mr. Johnson quotes from his tables that a slab 12 ft. span, 6 ins. thick, with 3-10-sq. in. of metal per running ft., is capable of carrying 615 lbs. per sq. ft. ultimate load, possibly the best way to prove the need of vertical reinforcement is to compare his floor with a public test recently made on one of the writer's floors in Washington, with vertical reinforcement, 16 ft. span, and the same thickness of concrete, but with 6-10-sq. in. of metal per running ft. This carried over an area of 62 sq. ft., a superimposed distributed load of 52,000 lbs., equal to 840 lbs. per sq. ft., without even the appreciable deflection of a hair. Reducing the conditions of this floor to the same span as that cited by Mr. Johnson, the Kahn floor should have carried, on a 12-ft. span, safely 1,500 lbs. per sq. ft., with 6-10-sq. in. of metal per ft. of cross-section, or with 3-10-sq. in. per ft., 750 lbs., without any appreciable deflection, as against 615 lbs. breaking load of the Johnson floor.

Some time ago the writer made a number of tests on small beams 2 ins. wide and 4 ins. deep, such as would occur in the section of a floor slab. Some were reinforced with plain rods along the bottom, Fig. 11; others with rods, as Mr. Johnson suggests, Fig. 12; and still others with the Kahn system, Fig. 13. The first two beams failed by vertical and longitudinal shear at respectively 3,000 lbs. and 3,150 lbs.; the Kahn beam at 4,850 lbs., the steel parting as usual near the center. All beams had been formed in the same mold and had the same cross-sectional area of steel.

As for the writer assuming "that all steel-concrete beams fail by shear," this has never been claimed. It would be quite ridiculous to assume, for example, that a beam 24 ins. wide by 36 ins. deep, 6 ft. span, with a single quarter-inch rod, should fail in this manner. Where such an abundance of concrete is given to a construction, or rather so little reinforcement is used in comparison with the large amount of concrete, the beam is hardly apt to shear. There are cases where properly proportioned beams, reinforced horizontally, have failed otherwise than in vertical or in longitudinal shear. These, however, are very rare indeed. Where beams have been economically designed with some practical idea as to proportion, and where loaded uniformly on top, so as to develop both shear and bending moment to the same extent, there are but few tests which show failure by the pulling of the steel apart. Regarding Prof. Howe's experiments, these have not as yet appeared in print; limiting ourselves, therefore, to tests of the past, which the writer believes were made to verify formulae advanced by engineers engaged in reinforced concrete, let us refer to table No. 4 on beams in the "Transactions of the A. S. C. E." of June, 1903. These tests were made under the supervision of Prof. Lanza, at the Massachusetts Institute of Technology, on 27 beams, 8 ins. by 12 ins., 11-ft. span, and reinforced in every conceivable way, with twisted and plain bars, and all proportions of reinforcement. A copy of the table is herewith published.

As stated in the table:

All the fractures except Nos. 1, 2, 3, 4, 19, 20 and 21, were of the following general character: They consisted of a longitudinal shearing break at a little above the reinforcing bar, and in addition, a break extending diagonally (often at an inclination of about 45°) upwards towards the center. The point where this diagonal break joined at the top of the beam was sometimes at the point of application of the load and sometimes near it.

Now, observe the construction of the 7 beams out of the 27 tested, which did not fail in accordance with the above general note. No. 1 was a beam without any metal reinforcement whatsoever. No. 2 had a rod of 1-16-sq. in. cross-sectional area imbedded in it. The writer has built beams of the same dimensions, using, however, 1 sq. in. of steel in the bottom, and developed practically the full strength of both the metal and the concrete, a test having 1-16-sq.in. of metal imbedded in the beam is therefore unscientific. No. 3 had ¼-sq. in. of metal imbedded therein. It seems that this quantity of metal produced a beam equally strong in bending and shearing, as the beam apparently failed in both ways at the same time. No. 4, concrete crushed on top. No. 19, beam failed apparently in vertical shear. No. 20 failed by longitudinal shear. No. 21, the reinforcement slipped.

Of 26 tests, with reinforcement, therefore, made at one of our most representative institutions, 23 beams failed by shear and only two by bending stress, and these two with a quantity of steel absolutely out of proportion to the area of concrete. And these results were obtained with the beams subjected to a load applied at 1 or 2 points, wherewith the shear was but slightly taxed in comparison with the bending moment. It will be well to note that in beams 12, 13 and 14, U-shaped stirrups were used, enclosing the rods, with very little effect in strengthening the beam. Is not this proof quite sufficient that beams do fail in shear rather than in bending moment? The writer would also call attention to the 12 tests made by one of our very important railway companies, and referred to in his recent article, all of which failed by shear. All had been reinforced with Mr. Johnson's corrugated bar and built in accordance with the directions in his catalogue. The writer does not care to mention the name of the company, since he has been requested not to do so.

As for the tendency of the steel to slip in the center: Let us again look at the photograph, Fig. 6, which appeared about a year ago, of a test slab supported on beams around its periphery, the construction being reinforced with twisted bars. Note the direction of the shear cracks and their closeness together. Does it seem as though the steel had very much of a tendency to slip at the center, or does it seem as if vertical reinforcement were necessary at this locality? It would have been well for Mr. Johnson to have studied this photograph before making his assertion regarding the necessity of providing against slipping at the center of the beam. Mr. Johnson has serious objections to the writer's bar because it is left unsheared at the center, and, therefore, as he claims, the concrete does not adhere to it at that place. This to the writer seems positively ridiculous. Mr. Johnson should have proceeded just a little farther and questioned why the enormous amount of reinforced concrete work so successfully erected in Europe, stands up at all, since only plain bars without his corrugation were universally used. But how would Mr. Johnson explain the test shown in Figs. 9 and 10, where the main reinforcement is entirely outside of the concrete, and only the prongs extend into it, where a beam 13 ins. wide, 11 ins. deep, with 4 ins. of brick on the outside and concrete between, carries on a 12-ft. span, 40,720 lbs. of steel billets with only ¼-in. deflection? Surely it points clearly to the fact that the writer does not depend much on the adhesion of the concrete to his main reinforcements, and that possibly there is some truth in the flat arch action in a reinforced beam after all. (For description of this test see Engineering News of Oct. 17.)

Finally, as for Mr. Johnson's statement that Kahn bars must be prepared in advance for a particular place. While this is not so in all cases, as shown above, does Mr. Johnson really think it such a serious objection that drawings and calculations should be carefully made for the steel-concrete construction at the office, in the same manner as would be done for steel construction? That the bars for each main girder are cut to exact lengths, marked and numbered, that each bar comes to the building a completed unit intended for and fitting a definite place? Does it seem an error that numberless loose stirrups need not be entrusted to careless workmen, or that bars need not be bent up at the building and set in place possibly at wrong angles, or where they do not belong? That high carbon steel rods need not be heated, burned and bent around the flange of a beam? Does it seem so impractical that nothing is left to the unskilled workmen at the building save the mixing of concrete and the placing of the one completed unit? Is not the fact that each main bar has been cut to length at the shop and distinctly marked for its own place in the building, sufficient to assure peace of mind for the engineer, and to bring him sound, restful sleep?

Referring to Mr. Schaub's remarks, the writer agrees with him in many ways, and feels that the errors in construction are possibly regarded from different points of view. It does seem, however, that depth in the design of beams should always be aimed at, as such depths add very materially to the strength. If a long span were in consideration, it hardly seems that a shallow beam would be consistent with good design. Of course, unless it is absolutely certain that the horizontal reinforcement is capable of taking the full longitudinal component of stress, shallowness is the only safeguard, as a deep beam with the proper amount of reinforcement arranged horizontally only, would undoubtedly shear. The only remedy, therefore, seems to be that of providing the reinforcement with such positive abutments throughout its length, that there is absolutely no question about the slipping of the bar, and to reinforce it vertically so that there is no danger of vertical shearing. The writer feels that he has accomplished this result in his construction. Where the diagonal bars are secured to the horizontal rod, absolutely positive abutments are formed. At the same time these diagonals tie the beam together vertically and carry the shear. As for the turning up of the bar at the ends, of course this lends efficiency to the beam and assists in preventing slipping at the ends. The difference between this method and the writer's is, that besides providing for slipping at the ends with both a positive shoulder, and a pair of bent-up prongs, he provides for it at distances along the rods equal to the depth of the beam, or in other words, referring to Fig. 3, and regarding again the stresses in the beam as internal arches, each interior arch finds an abutment at a, b, c, d, etc., and of course the stresses of these various arches are taken up within the tie-rods aa, bb, cc, etc. This latter method would, therefore, be similar to a self-contained flat arch, which, no matter how great the deflection of the beam might be, would still be in positive action everywhere, as the greater the deflection, the greater the clinching action between the various voussoirs. The end prong receives the horizontal thrust due to the end shear only, intermediate thrusts having already been taken up before reaching the end. With Mr. Schaub's method, however, if for any reason the end hook should give way, this receiving the sum total of the thrust of all arches, providing friction on the bar were disregarded, then the beam should collapse in longitudinal slipping, as already mentioned.

That there is much danger of the end of a rod, when bent up as suggested, straightening out when the beam is loaded, will readily appear if reference is made to tests by Sanford E. Thompson, published in the "Enginering Record" of Sept. 12, 1903. As for the writer's reinforcement, it absolutely can not slip. So many tests to destruction have been made on his bar without the slightest evidence of slipping that this has been definitely proven.

Julius Kahn.

Detroit, Mich., Feb. 6, 1904.

Notes and Queries.

Information is wanted concerning Patrick O'Connell, a native of Providence, R. I., and formerly engaged in contracting work. He left Providence in 1882 to take charge of some work on the Panama Railroad. If still living he would be about 50 years of age.

"L. E.," Chicago, Ill., writes:

I would be glad to know if there is an appliance in the market for boring holes through ice to be used in connection with soundings. I understand such a machine is

used extensively on government work in the vicinity of Duluth.

It would not be a difficult task, we should think, to rig a large size hand-auger with mechanism for rotating it and forcing it downward, or some of the hand power rock drills now on the market might fill the bill.

Our attention has been called to two errors in the article "Graphical Determination of Angles in Skew Structural Work" printed in our issue of Jan. 14, 1904. Near the close of the article, under the heading "Angle N." reference is made to the line K Z — K' L. The letter Z herein referred to was omitted from the diagram accompanying the article; it should have appeared at the intersection of line K W with C L. Again, at about the middle of the article, under the heading "Angle D," the sentence occurs, "A line normal to the hip web will revolve into H in a parallel to a C." This should read ". . . in a parallel to b C."

We printed last week a report of the meeting of Boston Members of the American Society of Civil Engineers to discuss the Union Engineering House project. Since that report was received a fuller statement of the ballot taken at the meeting has been sent us and we printed it, as follows:

The notice of the meeting was sent to 185 members, and in order to obtain an expression of opinion from as many of the members as possible those who were not able to attend the meeting were requested to express their opinion by letter. A vote was taken upon this proposition: "Shall the American Society of Civil Engineers become one of the Constituent Societies in the occupancy and control of the proposed Union Engineering Building, under the terms outlined by the Joint Conference Committee?" The result was as follows:

	Not present at the meeting.	Present.	Total.
In favor	25	46	71
Opposed	16	4	20
Not voting	84	10	94
Total	125	60	185

ILLINOIS ENGINEERS AND ILLINOIS ROADS.

By Ira O. Baker,† M. Am. Soc. C. E.

In Illinois there is just now a distinct call to the engineers of the state to help in finding the best solution of the good-roads problem. The legislature at its last session authorized the governor to appoint a commission to study the road question and report its conclusions to the next legislature. The commission has been appointed, and its report is sure to bring the road question up for active consideration. Is it too much to say that the civil engineers of the state, particularly the members of this society, scattered in the several communities, should contribute of their knowledge to the solution of this problem?

The good road problem is usually much more than a question of materials and the methods of using them. Matters relating to efficient road administration, and to the equitable adjustment of the expense of any proposed improvement between the lowest and the various higher governmental units, and a proper distribution of the expense between the present and the future, will require the wisest statesmanship, the most careful consideration of conflicting interests, and a recognition of the power of established practice. The engineer who has been trained to weigh causes and foresee results should be able to throw some light upon this part of the good road problem.

In the next legislature an attempt will probably be made to embody some view of road reform in legal enactment, and engineers should be ready to discuss the question with intelligence. I am not unmindful that at the annual meetings of our society there have been papers and reports upon various road matters; but such articles are written for engineers, while nearly all the rural roads are in name and in fact under the control of the farmers, and it is highly improbable that any material legal change will be made in road matters which is not approved by the farmers. The problem then is to induce the farmers to take better care of the present roads, or to instruct them in improved methods of caring for the present roads, or to persuade them to approve means that will lead to better roads than the present ones can ever be. It appears that there are many who are surprised to learn that anything can be done to improve the ordinary earth road.

One of the minor problems is to find a suitable material for the floors of highway bridges. At present, in this state, oak planks are generally, if not always, used; but it is now nearly impossible to find fairly satisfactory lumber for this purpose, and in the future the difficulty will increase. Not only is lumber growing poorer in quality and more scarce, but its cost is increasing rapidly. Apparently, concrete is the most promising substitute, but we lack data for its intelligent use. In the first place little or nothing is known about the shearing strength of concrete, and but little is known about its tensile strength. Of course, these data must be obtained by those having access to a testing machine. The University of Illinois has made a start toward the determination of what may be called the coefficients of concrete. The university has also begun some experiments directly upon the strength of concrete when employed for floors of highway bridges.

Another of these relatively minor problems that the public needs help upon is whether a broken stone road can be built in the corn belt of Illinois that will not be pulled to pieces by the mud brought upon it from the unpaved roadside? Some poorly constructed roads have been quickly pulled to pieces, which seems to show that there is at best only a narrow margin between success and failure. As the result of a single piece of road would not be conclusive, it is desirable that also this experiment should be made in different localities by different observers, the methods and results in each case being carefully recorded and published.

*Extracts from a paper read at the annual meeting of the Illinois Society of Engineers and Surveyors at Champaign, Ill., Jan. 21, 1904.

†Professor of Civil Engineering, University of Illinois, Champaign, Ill.

ANNUAL MEETING OF THE CONNECTICUT SOCIETY OF CIVIL ENGINEERS.

For the first time in its 20 years of existence, the Connecticut Society of Civil Engineers held a two days' session, Hartford being the place of meeting, and Feb. 9th and 10th the days of meeting.

This society has now a membership of 236, and fully 40% of this number attended the convention which in every way has justified the expectation that two full days may be annually devoted to the reading of papers, the discussion of previously printed papers and to social pleasure.

Hon. Ignatius A. Sullivan, Mayor of Hartford, delivered a cordial address of welcome, after which the report of the Secretary and Treasurer, Mr. J. Frederick Jackson, of New Haven, was read. During the year 1,200 copies of the nineteenth annual report were printed and distributed. Fourteen new members were received into the society, besides two honorary members. The receipts for the year were $1,729.69, and the disbursements were $1,106.35, leaving a balance of $623.34 on Feb. 8, 1904.

Mr. J. K. Punderford, of New Haven, was elected President, to succeed Mr. Frederick L. Ford. Mr. Ralph M. Wilcox, of Middletown, was elected Vice-President; Mr. Walter T. Spencer, of New Haven, was elected Second Vice-President; and Mr. J. Frederick Jackson was re-elected Secretary and Treasurer. President Frederick L. Ford then delivered the annual presidential address, which was presented in a novel form as a contract and specifications, in which the names of many of the members appeared, wittily arranged.

In the afternoon session of the first day, an address on "Good Roads" was delivered by Mr. James H. Macdonald, State Highway Commissioner. Before beginning his prepared address, Mr. Macdonald paid a tribute to the memory of the late James P. Bogart, of New York, who was an active member of the society. Mr. Macdonald stated, in the course of his address, that since the State of Connecticut took up the building of good roads in 1895, there had been expended $1,000,000 for which 500 miles of hard roads had been built. He estimated that in consequence of this object lesson, the towns have built 1,000 miles more of good roads without any assistance from the State. During the second day's session three interesting papers were read, the first of which was an exceedingly practical paper presented by Mr. W. Scott Morton, Chief Engineer of the New Milford Power Co., a corporation which has recently built a large dam and feeder canal on the Housatonic River. The dam itself is of rubble concrete in which 1¼ bbls. of Portland cement were used per cu. yd. The top and face of the dam are granolithic, the concrete used being 1 cement, 1½ sand, and 1½ screened pebbles; and after three months of use no abrasion has taken place. The dam is arched so as to give a greater factor of safety, although it is designed as a gravity dam. Its maximum height is 24 ft. and its length 203 ft. A large number of photographs and blue prints were used to illustrate the work in progress and when completed.

"Forestry in Relation to Water Supplies," was the title of a paper by Prof. H. S. Graves of the Yale Forest School. He discussed the economics of forest planting on the lands owned by water companies, and gave data relative to the work thus far done at New Haven and at Hartford.

The next and last paper was read by Mr. W. H. Richards, Superintendent and Engineer of the water-works system of New London, in which he described a storage reservoir built in the Oswegatchie hills. The new reservoir covers 40 acres and has a drainage area of 2.7 sq. miles.

The annual banquet was given in the large private dining room of the Hartford Club, and 115 members of the society were present. The menu cards were fac-similies of a contract and the requisite bond. Mr. Frederick L. Ford, City Engineer of Hartford, the retiring president, was toastmaster. At his right sat Mayor Sullivan, State Highway Commissioner James H. Macdonald, Hon. Martin Dodge, Director of the Office of Public Road Inquiries of Washington, D. C., and W. E. McClintock, Chairman of the Massachusetts Highway Commission. At his left hand were H. S. Earle, Highway Commissioner of Michigan; J. W. Hunter, Highway Commissioner of Pennsylvania; Joseph Buttis, President of the Street Board, and W. S. Morton, Chief Engineer of the New Milford Power Company. Those who responded to the various toasts were: Mayor Sullivan, Hon. Martin Dodge, Horatio S. Earle, Henry J. Kellogg, James H. Macdonald, A. P. Boller, W. E. McClintock and John K. Punderford.

MOVING A PARIS FOOTBRIDGE OVER THE SEINE AND WITHDRAWING ITS PIERS BY HYDRAULIC JACKS.

By Rene Bonnin.*

The northern section of the circular branch of the Metropolitan underground railway of Paris, which was opened to travel in March, 1903, continues across the Seine to the southern portion of Paris to join the Place de l'Etoile with the Place de la Nation, following the exterior boulevards. This last line, now under construction, after leaving the Trocadero station and passing underground beneath Franklin and Alboni streets, crosses the Seine where the Passy footbridge formerly stood. This footbridge, which was built in 1878 to give passage to pedestrians only, was neither sufficiently strong nor wide to allow of its being utilized for carrying the tracks of the Metropolitan, and to serve at the same time as a

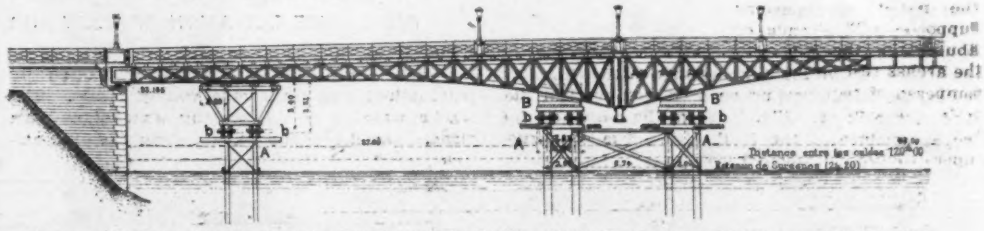


FIG. 1. PART ELEVATION OF PASSY FOOT BRIDGE MOUNTED ON TRESTLE TRACKS FOR MOVING.

wagon bridge connecting the two popular quarters of Paris, Passy and Grenelle. Hence it became indispensable to replace the Passy footbridge by a viaduct which should occupy its site. This viaduct has two stories; the first, or lower story to be used for teams, and the upper one for the trains of the Metropolitan. As, however, the travel between the two shores could not be stopped while the construction of the new bridge was going on, it was decided to preserve the footbridge temporarily and to displace it parallel to itself and about 100 ft. down-stream so as to have enough room for the construction of the new viaduct.

The Passy footbridge is composed of two parts (Fig. 9), one across the main channel of the

*61 Rue Louis Blanc, Paris, France.

Seine, on the Passy side; the other and smaller portion on the Grenelle side, the two parts being separated by the Ile des Cygnes. Each part of the bridge is divided into three spans. In the longer part the middle span is 48 inches (157.48 ft.) wide, and the side spans 37 m. (121.39 ft.) each. For the shorter part the spans are 34 m. (111.54 ft.) and 28 m. (91.87 ft.) respectively. The metallic portion which carries the deck is made of three longitudinal trusses of which the lower chord is curved, giving the appearance of an arch. The three trusses at each end of the middle span are connected by means of a bolt so as to form a joint or hinge. The total length of the long part of the bridge is 122 m. (400.26 ft), while that of the short part is 90 m. (295.28 ft.) The width of the bridge between the hand-rails is 6.50 m. (21.32 ft.). The ends of the metallic part are supported by masonry abutments; the ends of the arches rest on piers in the river. Each pier is composed of two metallic columns filled with concrete, the whole resting on a caisson sunk in the bed of the Seine. Fig. 1 shows the whole of the superstructure of the footbridge. The construction of the bridge is such that all the pressures on the supports are vertical, and there are no trusses.

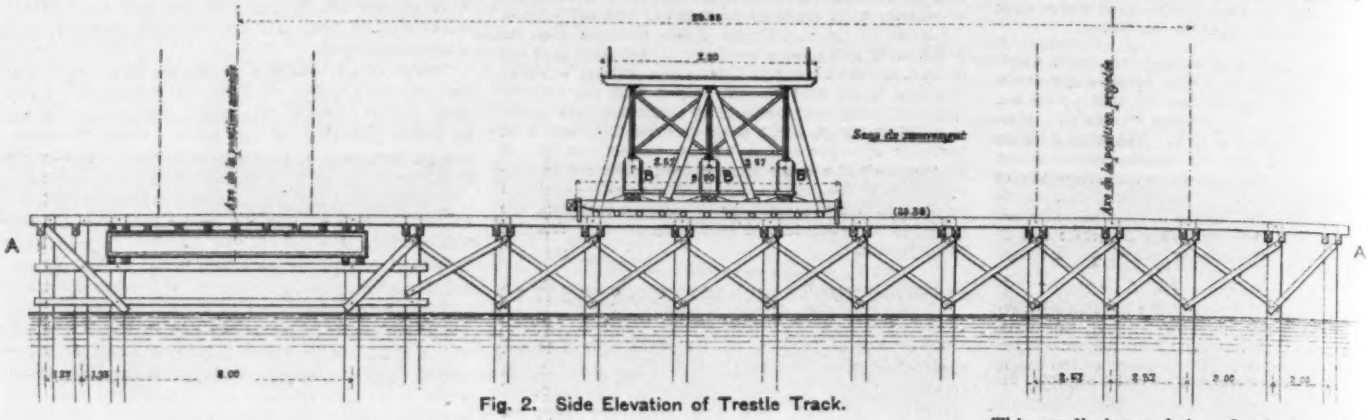


Fig. 2. Side Elevation of Trestle Track.

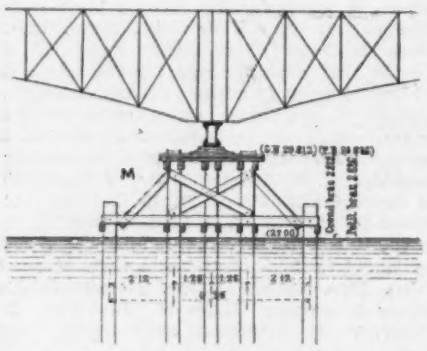


Fig. 3. Temporary Main Pier.

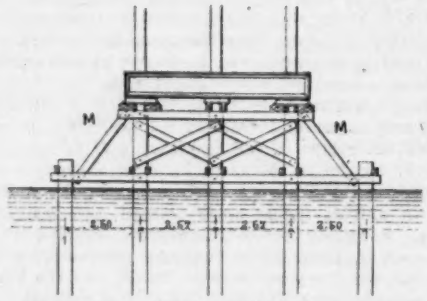


Fig. 4. Temporary End Pier.

FIGS. 2 TO 4. TRANSVERSE AND LONGITUDINAL ELEVATIONS OF TRESTLE TRACKS.

Two different methods were used to move the bridge. The whole metallic portion of the long section was moved over on rollers running on scaffolds, built parallel to the current of the river, and reaching from its old to its new position. The small section was moved on two scows, each carrying one half, and brought by means of windlasses to the new position.

LONGER SECTION.—(Figs. 1 to 4.) Tracks A for the rollers were first established on each side of the piers, and near the point of support on the abutments. The tracks in the river were wooden scaffolds, made of pile piers driven in the bed of the river and secured against any motion by longitudinal and transverse waling pieces. The tracks were to be 40 m. long, extending down stream parallel to the current from the old position of the bridge to a little beyond the new one (Fig. 2). Three wooden trestles, B, intended to support the three trusses of the bridge, rested on the scaffold. The trestles were shaped to the curve of the lower chord of the trusses. Rollers, b, were placed between the sills of the trestles and the top stringers of the scaffold. The track proper was an iron plate laid on the top stringer.

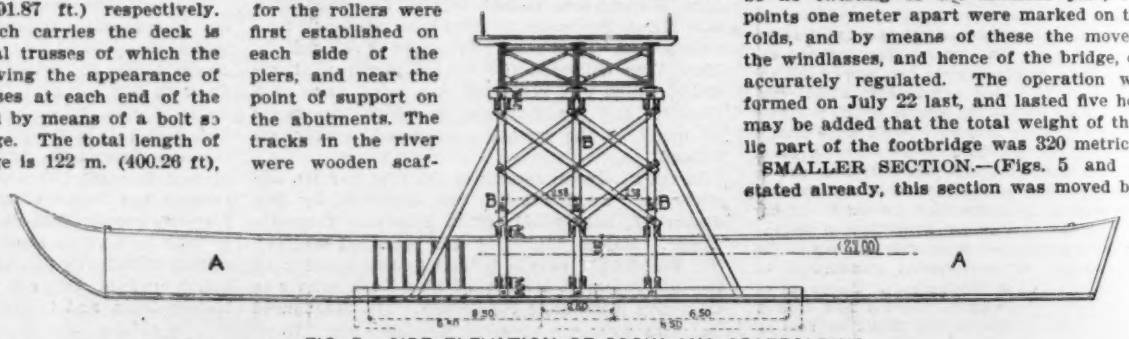


FIG. 5. SIDE ELEVATION OF SCOW AND SCAFFOLDING.

This preliminary being finished, and the metal cap of the pier being taken away, the whole metallic part of the bridge was lifted without even removing the joint at the middle of the center span, by four hydraulic 100-ton jacks, one to each column, which rested on the concrete in the columns. After the bridge had been raised, wooden wedges were driven in hard between the top of the trestles and the bottom of the chords, and secured firmly thereto by bolts, so that the entire weight of the metallic part should rest on the trestles. Then the bridge was gradually drawn to its new position by means of hand windlasses set on the scaffolds and lines attached to the trestles. In order to set the metallic part of the bridge on the wooden cushions prepared to receive it, it was raised a little by the jacks, the rollers were removed, and the bridge lowered to the cushions (MM Figs. 3 and 4).

In order to insure the moving of the bridge so that its different positions should be always parallel to the initial one, and so that there should be no twisting of the metallic part, reference points one meter apart were marked on the scaffolds, and by means of these the movement of the windlasses, and hence of the bridge, could be accurately regulated. The operation was performed on July 22 last, and lasted five hours. It may be added that the total weight of the metallic part of the footbridge was 320 metric tons.

SMALLER SECTION.—(Figs. 5 and 6.) As stated already, this section was moved by means of scows, each half being transferred separately. Three trestles, B, intended to carry the trusses of the bridge, were set on each of the two scows, A. These trestles were similar to those used for the larger section, but of somewhat different shape.

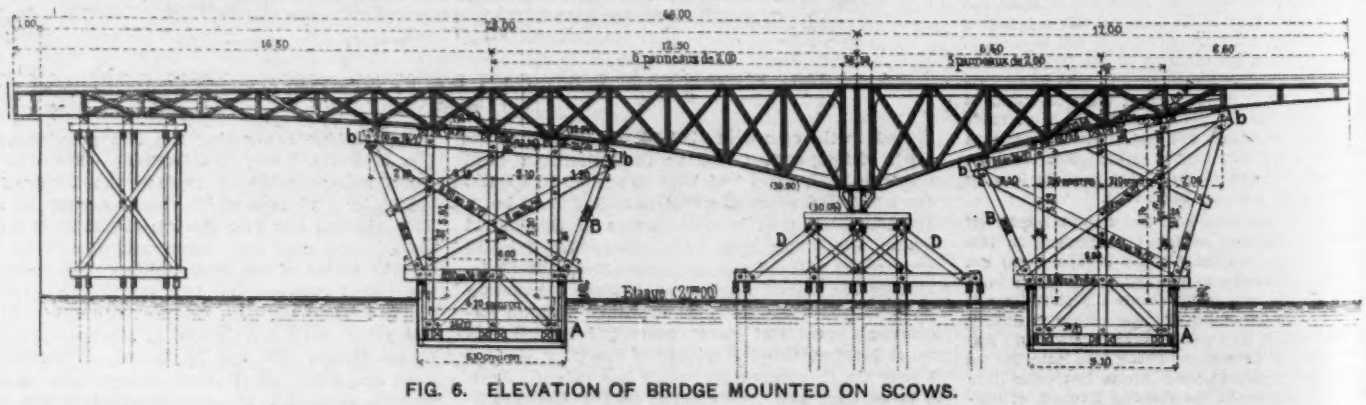


FIG. 6. ELEVATION OF BRIDGE MOUNTED ON SCOWS.

The scows, having first been ballasted so as to leave a space of about 15 cm. (6 ins.) between the top of the trestle and the bottom of the chords, were brought in, one on each side of a

pier. The scows were secured by means of anchors, and ropes fastened to the piers. Wedges b b were driven into the space between the trestles and the chords, to both of which they were fastened by bolts. Then, after the joint, M, of the middle span had been removed, the ballast was removed from the scows. As the ballast was taken out the scows rose gradually and lifted the bridge from its bearings. The operation was continued until everything had been cleared by about 20 cm. (8 ins.), when the scows were moved to the new position of the bridge by means of the lines attached to the anchors. The half-length being placed accurately, the ballast was replaced in the scows until the structure came down to its new bearings on the temporary piers, D, which had been built beforehand, and the wedges could be removed. The scows being freed came back for the second half, which was moved in the same way as the first. The junction of the two halves, by replacing the bolt of the articulation, was accomplished through the cross movement of the scows on the lines which attached them to the temporary piers.

The weight of the metallic part of each half of bridge was 110 metric tons, of which 37.62 tons was carried on the scow under the center span, and 72.38 tons on the one under the shore span. The operation was performed, for the first half, on Aug. 20; and for the second half, on Aug. 21. In spite of a heavy wind blowing on each day, each part of the bridge was moved in 5½ hours.

REMOVAL OF THE PIERS OF THE FOOTBRIDGE.—The viaduct which is to replace the footbridge has two stories, the lower one for a driveway and the upper for the Metropolitan railway. It, too, is composed of two sections separated by the Ile des Cygnes. Each section rests (Figs. 7 and 9) on two piers in the river, the distance between their centers being for the main arm 54 m. (177.16 ft.), and for the secondary arm 42 m. (137.79 ft.). It happens that two of these piers occupy exactly the positions of those of the footbridge. As it would have been very difficult under these conditions to put down the new piers by compressed air, as they not only rise above the old ones, but also must go much further down to reach a firm bottom, it was decided to remove the old piers and their supporting caissons as a necessary preliminary to any new work. The old piers might have been blown up, as is the ordinary way of doing such work. It was preferred, however, to adopt another and very ingenious expedient, which, so far as the writer knows, has never yet been applied, at least under similar conditions. The method adopted was to take out in one block the caisson and the columns of the old pier, by raising them with hydraulic jacks. The mass to be raised, as shown by Fig. 10, is composed of the two columns and the caisson, founded on the clay stratum at a depth of 3.50 m. (11.48 ft.) below the bed of the river. Its weight is 260 metric tons. Besides this weight provision had to be made for the friction of the clay and gravel bed of the Seine against the sides of the caisson, even admitting the subpressures acting under the caisson. A very great amount of this friction, requiring an enormous lifting force at the outset, was to be feared. Happily, these fears were not realized. The subpressures and the friction seem nearly to have balanced each other, and, as a matter of fact, the effort of the jacks was but little more than the weight of the mass to be raised, that is to say, a maximum of 300 metric tons.

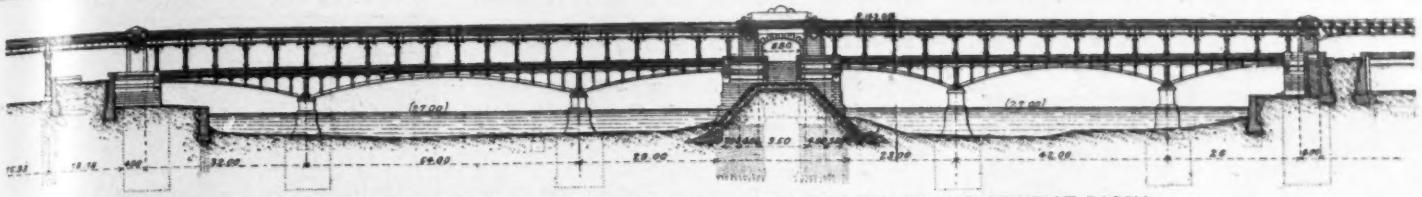


FIG. 7. PART ELEVATION OF NEW DOUELE-DECK VIADUCT OVER THE SEINE AT PASSY.

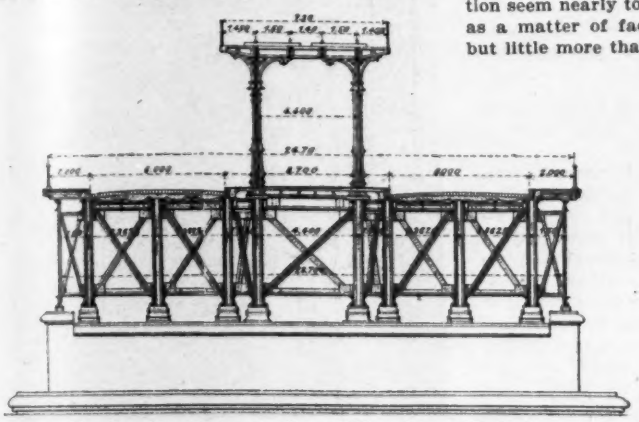


Fig. 8. Transverse Section of New Passy Viaduct.

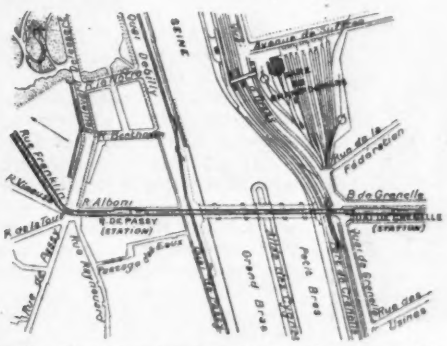


Fig. 9. Plan Showing Location of New Passy Viaduct.

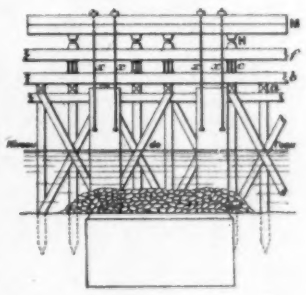


Fig. 10. Sketch Showing Plant for Pulling Piers of Passy Footbridge.

The operation was performed as follows: Caps a a were first laid on each of the tracks (Fig. 10) which had served for moving the bridge. Longitudinal wooden beams b were set on these caps. Then iron beams c rested on these longitudinal pieces. Very strong wooden beams f were placed on the iron beams, and on them stood the hydraulic jacks H, which were to do the lifting. There were eight of these jacks, each of 100 tons lifting power. They acted, in lifting, on a very stout beam, M, to which were bolted the iron rods x, which have their lower end fastened by stirrups to the walls of the columns of the pier to be raised. The stroke of the jacks being only 15 cm. (6 ins.), wooden blocks were placed, after each lift corresponding to this stroke, between the bottom of the jacks and the beam, f. Furthermore, care was taken as the pier was raised to fill the void under the caisson with stones which were forced under its sides.

When the caisson had been raised 3.50 m. (11.48 ft.) and its bottom reached the level of the bed of the Seine, the entire hoisting plant was taken down. The pier was then taken up, by ballasted scows placed on each side of the caisson, and carried ashore where it could be broken up.

This operation, as to the success of which there might well have been doubt, and for which no precedent could be quoted, deserves to be mentioned. In spite of its success, it might not be prudent to admit the generalizing of this method. It might happen, in certain cases, that, by reason of buckling or of the rough surfaces of the sides of the caisson, or by reason of the condition of the ground in which the caisson is sunk, the conditions would be altogether different from those at Passy, and that the resistance due to friction might make the work very costly, if not impossible. In such a case another solution would be preferred. Fig. 11 shows the whole of the plant.

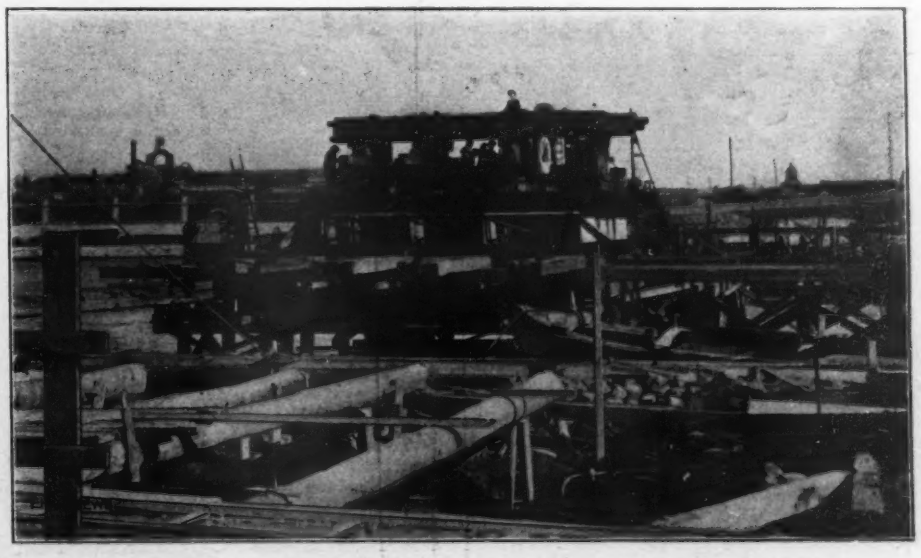


FIG. 11. VIEW OF PLANT FOR PULLING PIERS OF PASSY FOOTBRIDGE.

A NOVEL CENTER AND FORM FOR CONCRETE SEWER WORK.

The accompanying illustrations show a novel form of centering used in constructing some 800 ft. of 5-ft. egg-shaped sewer at Washington, D. C.

This particular sewer had a brick-lined invert, but the centering is designed especially for all concrete sewer construction, and has been used for sewers as small as 8 ins. and 12 ins. in size. It can be used to the greatest advantage, however, for sewers of 3 ft. and upward in size.

The view, Fig. 1, gives the best idea of the construction of the centering. It consists of two parts, an invert form and an arch center, and each is 16 ft. long. Both parts consist of a longitudinal lagging nailed to plank ribs spaced 2 ft. apart. To prepare the centering for use the arch center

Fig. 1. View of Steel-Wrapped Centering In Position for Concreting.

is set on top of the invert form, with the wedge timbers A and B between, and the two are locked together by latches on the outside of the end ribs. When erected as described, a square gudgeon timber is passed lengthwise of the centering through the hole C in the ribs. This timber projects at each end and is rounded so that the centering can be mounted on bearings carried by two horses, as is clearly shown by Fig. 2. A crank arm applied to one end of the gudgeon timber then enables the center to be rotated so that a strip of steel 6 ins. wide and 1-24-in. thick can be wound spirally around it. Figs. 1 and 2, both show the centering with the steel strip in place.

The centering with the steel strip in place is set in the trench and the concrete is filled around it to the proper thickness. As soon as the filling is completed, the wedge timbers are driven in until the ribs drop into notches. This allows the upper and lower parts of the centering to close together. They are then withdrawn, leaving the spiral strip of steel in place to carry the weight of the concrete. The centering is then wound for another section of work. After the concrete has set the steel strip is removed by simply pulling on the loose end. This work can be easily performed by one man, and the view, Fig. 3, shows one man doing it and coiling up the strip as it is freed from the concrete. To prevent the steel from adhering to the concrete, it is smeared with oil before the concrete is deposited. In the work illustrated black petroleum oil was used with good results, but the most satisfactory material was found to be linseed oil.

In the work at Washington about 200 ft. of 5-ft. sewer were constructed in an eight-hour day. The centering illustrated has been patented by Mr. A. C. Chenowith, of 16 Court St., Brooklyn, N. Y. We are indebted to Mr. Chenowith for the information from which this description has been prepared.

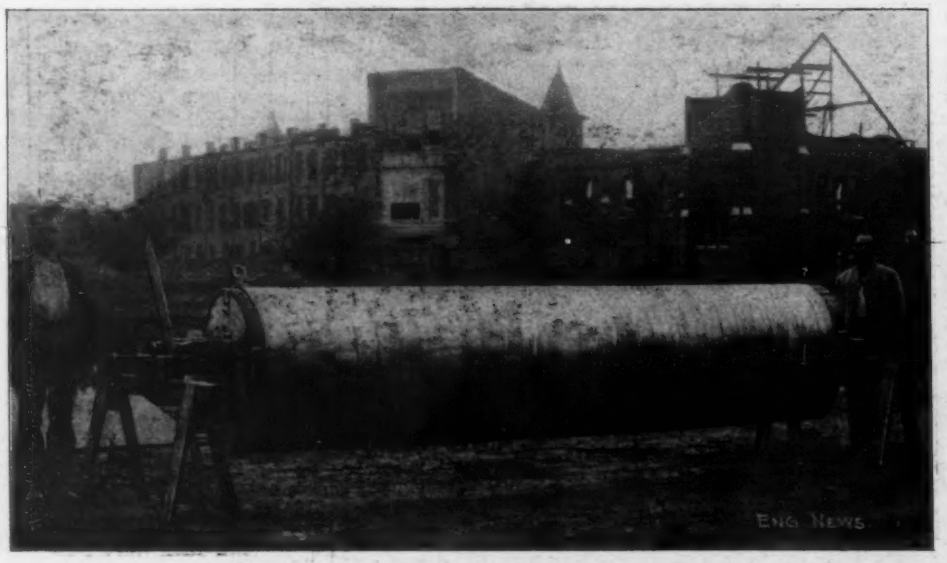


FIG. 2. VIEW SHOWING CENTERING MOUNTED ON BEARINGS FOR APPLICATION OF STEEL WRAPPING.

A CRUSHING AND CLEANING PLANT FOR GRAVEL BALLAST.

By far the greater proportion of railway track is ballasted with gravel, which term covers a very wide range of material, of varying quality or value for the purpose intended. As a general thing the gravel, whatever its quality, is excavated from the pit, loaded on cars, and put directly into the track, although the gravel almost always contains a considerable proportion of undesirable material. A few railways have experimented with the screening and washing of gravel, with good results. As the increase of traffic and the introduction of improved methods of operation lead to a demand for better materials and less work and cost of maintenance, the ballast will necessarily receive attention. Crushed stone and good clean gravel are both satisfactory materials, and clean gravel of uniform size is quite equal to good crushed stone. In this connection we quote the following remarks from Tratman's "Railway Track and Track Work":

Gravel ballast is used more than any other material, and is of very varying quality. It may be coarse and clean, or sandy and dusty, or loamy and cementing (when weeds will grow, drainage will be affected and the track will heave), or it may be full of large stones, in which case it will make an irregular and rough riding track. Cementing gravel should be avoided as far as possible. The best gravel is clean and coarse, with stones of fairly uniform size, and as little sand as possible. Only very clean gravel will drain as well as stone. It is good economy to use plenty of gravel, giving at least 8 ins. (or better, 10 ins.) under the ties, as this will enable a fairly good track to be maintained nearly all the year through without excessive work. It can be tamped by picks or bars, the latter being usually preferred, and is easily taken care of. Sandy gravel (like sand) may cause undue wear of tires, journals, etc. In fact, the substitution of slag for such gravel ballast on the Mobile and Montgomery Division of the Louisville & Nashville Ry. is said to have reduced the wear of engine tires from 1-16-in. per 2,000 miles to 1-16-in. per 16,000 miles. Screened gravel is used to some extent on the Pennsylvania Lines and the Lake Shore & Michigan Southern Ry., owing to the fact that the gravel as it comes from the pit contains 30 to 70% of sand, so that it is difficult to maintain the track

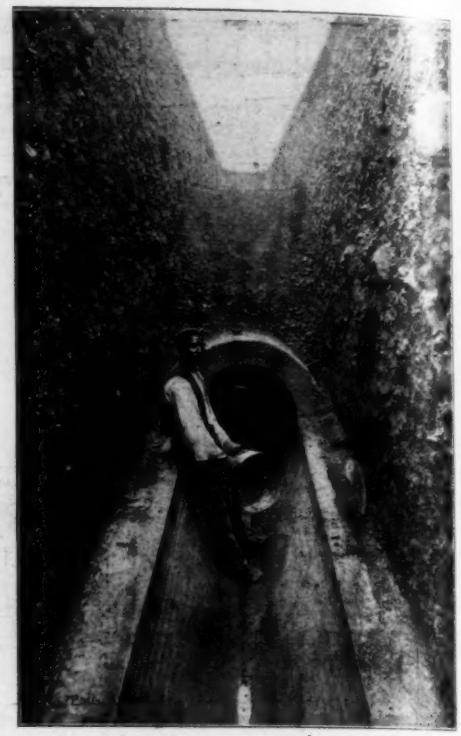


Fig. 3. View Showing Removal of Steel Wrapping from Sewer After Concrete Has Set.

in surface, the rains causing continual settlement by washing out the sand. The gravel is thrown into a hopper whence an elevator carries it to a revolving screen. In Europe, gravel is sometimes thoroughly washed and screened by machinery to free it entirely from earth and sand.

These remarks serve as an introduction to a description of a new plant for handling gravel ballast. This plant excavates the gravel in the pit, crushes, screens and washes it, and delivers the product directly into the railway cars. One of these plants has been in use by the Garden City Sand Co., of Chicago, for two years, and others have been in service recently upon the Chicago & Alton Ry. and the Kansas City Southern Ry. The plant has been invented and patented by Mr. M. E. Pugh, engineer and contractor, who has established offices at 89 South Canal St., and is preparing to introduce the plant on a number of roads. In many cases an investigation of gravel pits is necessary, as managers sometimes state that no suitable material for ballast exists on their lines, while investigation may show ample quantities of material which may be made suitable by proper treatment. Not only ordinary gravel, but also trap rock and loose flint deposits make good ballast when crushed and washed. As to ordinary gravel, that obtained in different parts of the country contains from 40 to 70% of sand, or 30 to 50% of a mixture of sand and clay. All this must be removed, and the remaining stone crushed and screened to a uniform size before it can be considered as a really good ballast material.

Coming now to a description of the plant, which is shown in the accompanying cuts, it consists of three principal parts: (1) A self-propelling steam shovel mounted on one car; (2) a revolving washer with sizers and crushers, located on a second car; (3) a pumping station at some convenient stream or pond from which a sufficient flow of water may be pumped for washing. The plan of operating is to lay two parallel tracks the entire length of the gravel pit, the inside track for the ballast plant to operate on, and the second track for the ballast cars to remain on while being loaded. If

the sand from the gravel is to be saved, a third parallel track is laid for holding the sand cars while they are being loaded at the same time. The system of operation is different from that of ordinary steam shovel work in gravel pits in that,

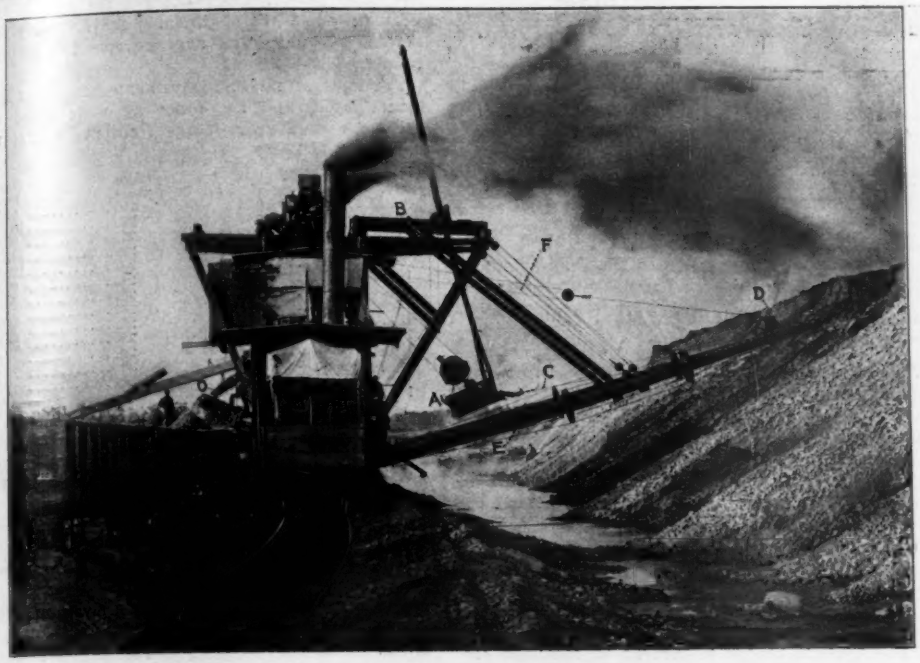


FIG. 1. END VIEW OF GRAVEL BALLAST PLANT ON THE CHICAGO & ALTON RY.
M. E. Pugh, Chicago, Inventor.

while the cars are being loaded, they remain stationary, and the machine moves along on the inside track, commencing at one end of the train and loading each car as it goes. Thus the expense of keeping a locomotive to spot the cars at the steam shovel is done away with. For this purpose the machine is provided with powerful moving gear connected to the car axles.

The dipper, A, works from the side of the car, and the dipper handle is carried by a frame, B. The hauling cable, C, for the dipper passes around the sheave, D, at the head of the boom, E, this boom being adjustable so that it can be lowered as the material is excavated. The suspension cables of the boom are shown at F. The dipper handle is provided with a rack and crowding engines which force the dipper into the material while making a cut. In Fig. 1, the dipper is returning to the car with a load, which is dumped into a hopper at G, Fig. 2, the dipper being automatically unlatched. The steam shovel is mounted on a 50-ton steel flat car, fitted with air brakes and a locomotive air pump, which operates not only the brakes, but also the levers controlling the dipper and the movements of the plant.

The material dumped into the hopper at G is fed into a bucket elevator which raises it to H, where it is dumped into hopper J. From this it is delivered to the rotary screen K, while at the same time a stream of water enters from the pipe L. The first section of the screen, at the right, has ¼-in. meshes, through which the sand, earth and water escape, and are carried off through a chute and discharged at M, as waste. If the sand is sufficiently good for building purposes, or for concrete, it may be delivered to railway cars. The right-hand end of the screen has a mesh of proper size for stone for ballast or concrete. Stones which are too large to pass through the walls of this screen are discharged at the end and fall to a crusher, N. The stone from the crusher, as well as that which comes through the screen, is delivered to another bucket elevator which raises it to the head of the chute by which it is delivered to the railway cars, as shown at O, Fig. 1.

The gravel bank shown in Fig. 1 is 30 ft. high and when the machine first started operation, the track was close up to the toe of the bank. For starting, there was first used a pair of A-frames 30 ft. long, and as the bank was worked out, they were changed to a length of 55 ft., as shown. By this plan the machine was able to make a cut 35 ft. wide, the entire length of the pit, without throwing the tracks. When the machine has made a cut the full width, the outside track is taken up and relaid between the machine and the bank, then the machine is moved over onto it, allowing the original machine track to be used for the ballast cars to stand on. If extra track material is provided, a new machine track can be laid in one end of the pit while the machine is working in the other end. Then the machine is moved over without the loss of time in taking up and relaying the outside track.

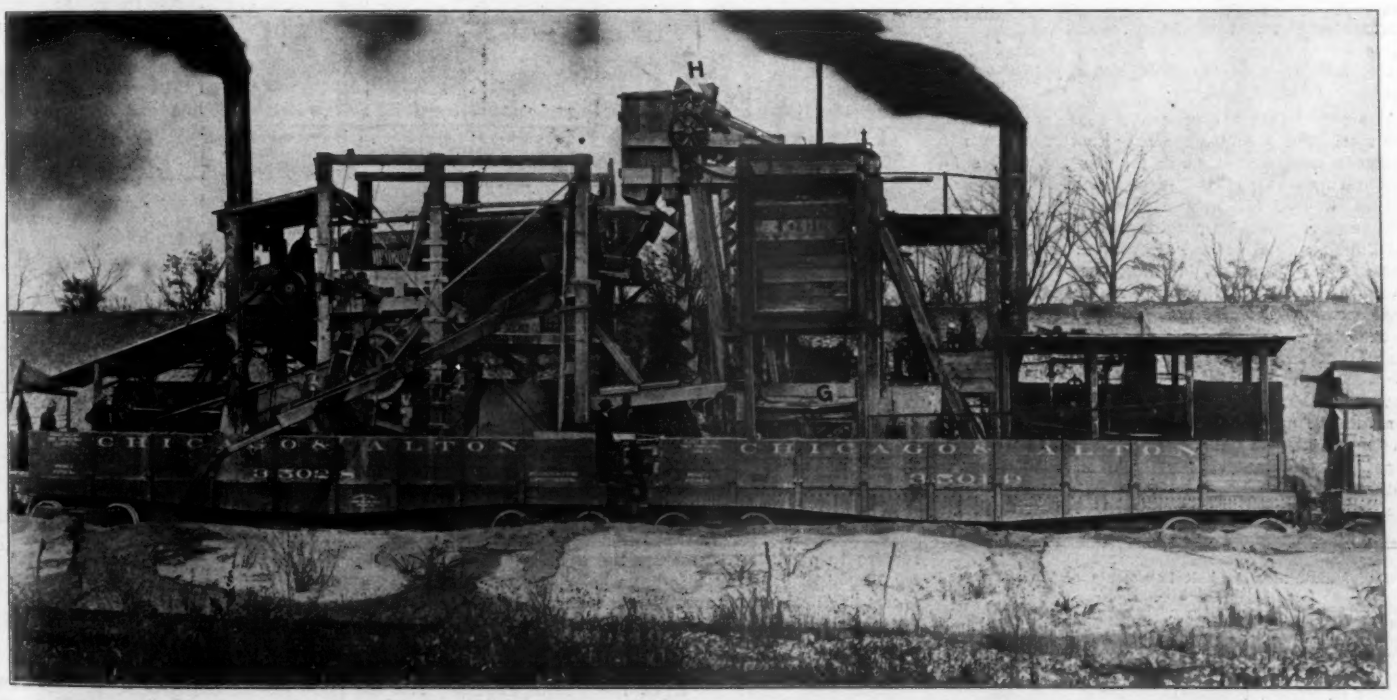


FIG. 2. SIDE VIEW OF GRAVEL BALLAST PLANT ON THE CHICAGO & ALTON RY.

MAINTENANCE OF ASPHALT PAVEMENTS IN WASHINGTON, D. C.

By Capt. H. C. Newcomer.*

Repairs to asphalt roadway pavements in Washington, D. C., are made under special contract for this class of work. While most public works are executed under annual contracts, it was found desirable to have this contract cover longer periods, and so authority was obtained from Congress to make the contract for any term not exceeding five years. It is customary to invite proposals for several periods within this limit, and then to award the contract for the term that appears most advantageous. The contract terminating June 30, 1903, was with the Cranford Paving Co. for a period of three years. The current contract is with the Brennan Construction Co., and runs for four years.

The total area of sheet asphalt and coal-tar pavements in the District of Columbia, exclusive of that maintained by the street railway companies, was about 2,886,786 sq. yds. on July 1, 1903. About 461,054 sq. yds. of this area were under five years of age and are maintained by the contractors that laid them, under the five-year guarantee, whose cost is included in the original contract price. The remaining area of about 2,425,732 sq. yds. is maintained by the city under the repair contract out of the appropriation for

*Corps of Engineers, U. S. A., Assistant to Engineer Commissioner, District of Columbia, Washington Barracks, D. C.

"Repairs to Streets, Avenues and Alleys." This fund has amounted to $200,000 per annum for several years past. It provides for considerable work besides the asphalt repairs, however, so that the amount available for this purpose does not exceed about $150,000 as a rough approximation. Of this sum about $100,000 are expended for entirely resurfacing or reconstructing worn-out pavements, and about $50,000 for minor repairs or patching. The relative proportions expended for resurfacing and repairs will, of course, vary with the exigencies of the work. Both classes of work are included under the same contract.

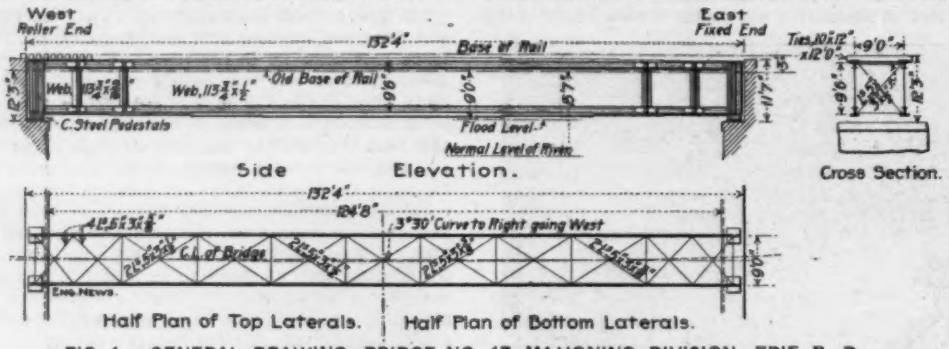


FIG. 1. GENERAL DRAWING, BRIDGE NO. 17, MAHONING DIVISION, ERIE R. R.
M. R. Strong, Bridge Engineer; American Bridge Co., Builders.

The method of resurfacing varies with the circumstances. In the rare case where the old pavement has a concrete base of proper form it is only necessary to remove the old top and binder and put on new. In the majority of cases, due to defective grade or insufficient crown, it is found necessary to remodel the base to a greater or less extent. The portions that are too high to permit the use of binder at least 1 in. thick are cut out and replaced by new base. The portions that are too low are built up either by using additional thickness of binder in case the deficiency is slight, or by using bituminous base, i. e., broken stone covered with coal tar. Our standard type of sheet asphalt pavement has a 6-in. concrete base, 1½ ins. of asphalt binder, and 1½ ins. of top or wearing surface after compression. Where the old pavement has a bituminous base, however, as frequently happens, it is usually retained in resurfacing, but is sometimes replaced by a concrete base. Vitrified block gutters, about 28 ins. wide, laid on concrete base, are supplied in all cases.

The pavements are generally kept in repair by the method of patching until the cost of such work becomes excessive, though on account of insufficient funds to pay the larger expense of resurfacing it is frequently necessary to repair defective pavements by patching after the economical limit for this method of maintenance has been passed. The work of patching goes on with one or more gangs, usually two, at all seasons of the year except when interrupted by rain or snow, or occasionally by necessary repairs to the asphalt plant that suspend its operation. Inspectors employed by the District of Columbia mark on the pavement the outlines of the defective spots. The contractor's men cut away and remove the defective material, and paint with asphaltic cement the edge of sound material; so as to give good bond with the new binder and top, which are placed in the hole and properly compacted. No attempt is made to plot these patches or determine their area, as the binder and top are paid for by the cubic foot. Continuous inspection at the asphalt plant and on the street, together with laboratory tests of material used, are relied upon to secure good work.

The prices under the current contract are as follows:

For standard asphalt pavement (2½ ins. asphalt surface and 2 ins. binder, before compression, on 6-in. concrete base made of 1 part Portland cement, 4 parts sand, 5 parts gravel, and 5 parts broken stone), $1.59 per sq. yd.

For standard asphalt surface (2½ ins. before compression), 67 cts. per sq. yd.

For standard asphalt surface (measured in cart), 49 cts. per cu. ft.

For asphalt binder (measured in cart), 25 cts. per cu. ft.

All items of additional work, such as grading, curb-setting, laying vitrified block gutters, repairing sidewalks, etc., are paid for at prices fixed in the specifications.

The printed specifications cover about seven pages, cap size (8½ × 14 ins.).

In view of certain statements as to the cost of repairs to asphalt pavements, the available data for Washington pavements have been analyzed to ascertain the average annual cost per square yard for maintenance of pavements of different ages. Some of the older pavements could not be included, as the earlier records were incomplete. It will be seen, however, from the accompanying table, giving the results of the investigation, that figures have been secured for quite a wide range of ages, with considerable areas in most cases.

This table does not give the cost of repairs during the five-year guarantee period, nor does it include the cost of renewal when the pavement is worn out and entirely resurfaced or relaid.

The work of tabulation and the computations necessary to arrive at these results were very laborious, and some errors may have crept in. The work was carefully done, however, by Mr. Geo. H. Bailey, Assistant Engineer, and the results are believed to be substantially correct.

Cost of Maintaining Asphalt Pavements of Various Ages at Washington, D. C.

Age in years.	Area, sq. yds.	Cost of repairs for the year.	Average cost per sq. yd. per year.
5	1,841,435	$11,897	$0.0065
6	1,809,869	13,965	.0077
7	1,747,461	31,385	.0180
8	1,653,811	38,531	.0233
9	1,597,313	42,871	.0269
10	1 476,575	38,500	.0260
11	1,292,200	43,003	.0333
12	1,068,848	42,270	.0396
13	913,795	31,546	.0345
14	804,420	28,435	.0354
15	698,826	21,576	.0309
16	608,117	23,479	.0386
17	560,523	18,913	.0338
18	504,995	23,012	.0456
19	374,800	11,951	.0319
20	272,040	7,182	.0264
21	192,643	3,870	.0201
22	104,001	2,887	.0280
23	36,332	678	.0187
24	35,647	1,268	.0356

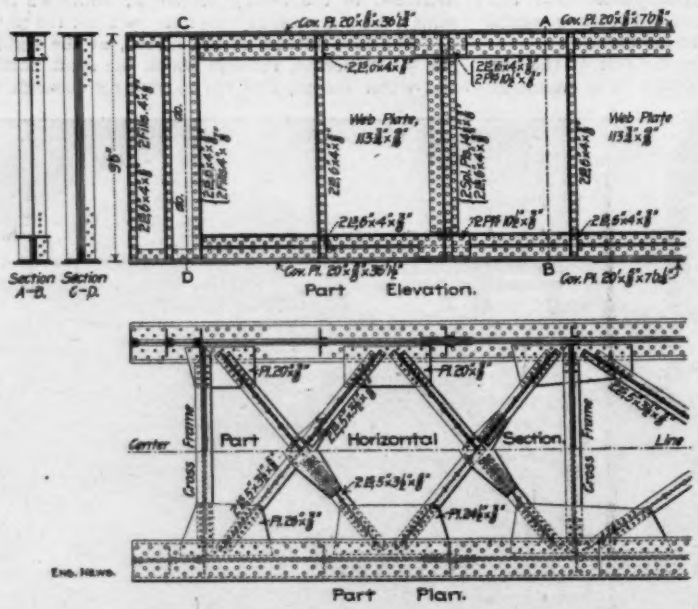


FIG. 2. DETAIL CONSTRUCTION OF BRIDGE.

The results are believed to be substantially correct.

The average cost per square yard per annum for the second five-year period of the life of the pavements considered was 1.65 cts.; for the third five-year period, 3.37 cts.; for the fourth five-year period, 3.78 cts., and for the fifth five-year period, 2.56 cts. The average cost for all ages tabulated was 2.8 cts.

The ages of existing pavements have also been investigated, and the following table gives the areas for each year of age from five up to the date July 1, 1903:

Age, years.	Square yards.	Age. years.	Square yards.
5	97,642	19	60,967
6	99,967	20	108,285
7	81,497	21	95,762
8	109,128	22	106,439
9	105,693	23	136,657
10	101,296	24	93,949
11	130,745	25	75,417
12	209,632	26	21,869
13	202,134	27	15,041
14	165,746	28	50,682
15	59,668	29	1,642
16	97,607	30	23,254
17	70,841	31	7,330
18	45,154		2,277,144

The average age of the areas shown is about 14.8 years, and there are over 700,000 sq. yds. of pavement that are over 18 years of age. The average age of the areas resurfaced during the fiscal year ending July 1, 1903, was slightly over 21 years.

It is a quite remarkable fact that the average age of all pavements being maintained under the repair contract is greater than the average length of life assigned to asphalt pavements in most cities.

TWO LARGE PLATE GIRDER RAILWAY BRIDGES.

The limit of span for plate-girder bridges has been increasing continually ever since engineers first developed the correct principles in the design and construction of plate girders. The limit is one of practicability rather than one of economy; ease of handling and erection favors the truss bridge, while, of course, the truss bridge will usually be the cheaper to build, when the span exceeds a certain "economic limit." This limit for railway bridges, is rather generally considered to be in the neighborhood of 100 ft. However, ease of handling and first cost are not the only factors entering the problem; length of natural life, cost of maintenance, safety against derailments and similar accidents, are some of the considerations which influence the design of railway bridges, and judged by these the plate-girder bridge has the advantage when the more obvious factors (first cost, handling, erection) do not clearly decide the selection. There are to-day a considerable number of plate-girder railway bridges of span exceeding 100 ft. in service in this country, and their number is steadily increasing. Two recent additions to the list are described and illustrated in the present article.

Bridge No. 17, Mahoning Division, Erie R. R.

This bridge, a deck plate-girder structure, carries the main line of the Erie R. R. over Yankee Run, near Hubbard, O. The line is single track, and over the crossing it is on a 3° 30′ curve. The bridge was built and erected during the past year. Previous to that time the crossing was made by a through truss bridge of the same span as the present bridge, but the old trusses had become too light for the traffic and required replacing. It was desired to make the new bridge a deck structure, because of the curve in the track, but the height from rail to high water level, only 9 ft., would not permit of this. Since the bridge

lay in a sag of the track, however, it was finally concluded to raise the rail level at the bridge by 3 ft., thus reducing the grades on either side and at the same time affording sufficient height for a shallow deck bridge. These conditions called for a plate-girder structure. The main girders are among the longest and largest plate girders ever

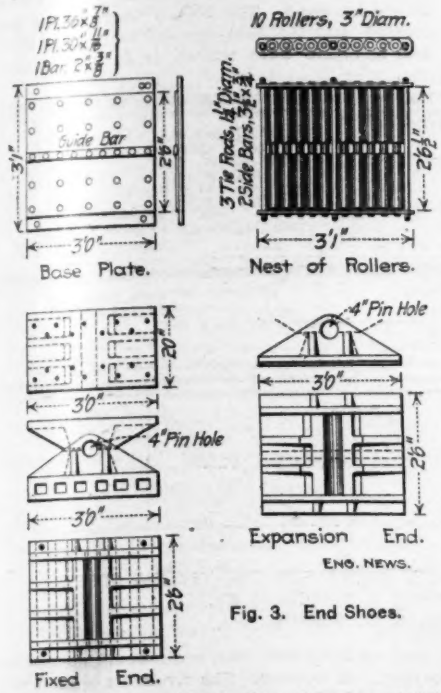


Fig. 3. End Shoes.

constructed. They were built and transported to the site as complete girders.

The general design of the bridge is shown in Fig. 1. The two main girders are 131 ft. 4 ins. over all, 9 ft. 6 ins. deep over backs of flange angles, and spaced 9 ft. apart transversely. The cross ties rest directly on the upper flange. Full top and bottom laterals are provided, with a panel length of about 14 ft. A sway-frame is used at each panel point. The end shoes are of cast steel, with rocker pins 4 ins. in diameter; the west end has rollers under the lower pin shoes. The steelwork of the bridge weighs 279,930 lbs.; each of the two main girders alone weighs 113,700 lbs.

The details of the main girders and bracing are shown sufficiently in the part drawing, Fig. 2. The end reaction from the shoe is taken up by three pairs of vertical stiffener angles. The web of the girder, 113¾ ins. wide, is 9-16-in. thick in the end sections and ½-in. thick in the center; it is spliced at every second stiffener (about every 12 to 14 ft.), in the manner shown in Fig. 2. Special attention is called to the reinforcing plates. 10½ × ⅜ × 26⅝ ins., at the flange end of each web splice; these are a concession to the function of the web in taking bending moment, in accordance with the following paragraph from the railroad company's specifications for bridges:

Plate-girders shall be proportioned upon the supposition that the bending or chord stresses are resisted entirely by the upper and lower flanges, excepting in girders of 90 ft. long and over, where one-eighth of the web shall be considered as flange section, and proper web-splices used. In girders less than 50 ft. long the cross-section of the flanges shall be assumed as consisting of only the horizontal legs of the angles, plus the cover-plate or plates. In those of 50 ft. long and over, the whole angles shall be counted.

The flanges are, necessarily, extremely heavy. Four 8 × 6 × ⅞-in. angles are used in each flange, in the I-shaped arrangement shown. Each flange angle is in two sections, which for the different angles vary in length from 52 ft. to 73 ft., approximately. The splices are arranged so that no two angles in one flange are spliced at the same point. The cover plates of each flange increase from one 20 × ⅝-in. plate at the end to four such plates at the center. The splices in these plates are made near the theoretical ending of an overlying cover, and the latter is extended a short distance beyond this point to carry the stress over the joint. A note on the railway company's stress sheet which we have omitted in Fig. 1 gives instructions for splicing cover plates, as follows:

Where necessary to splice cover-plates, do it by extending the outer ones over the lengths here shown (the theoretical lengths.—Ed.) and let the added length in each case run at least 4 ft. beyond the splice. Do not stop any cover just short of a web-splice. Carry it over the splice.

The detail arrangement of the cross-frames, which we have not shown in our Fig. 2, presents only this special feature: the gusset plates by which the cross-braces connect to the stiffener angle of the girder are notched around the outstanding leg of the inner flange angle. It will be seen from Fig. 2 that the stiffeners are not doubled at the cross-frames as is sometimes done.

The end shoes of the bridge are of cast steel. Fig. 3 shows their construction. The lower shoe at the fixed end is made extra high so that with a 1½-in. base plate and 3-in. rollers at the roller end the two shoes are of same total height. As shown in the figure, the rollers have a groove 2 ins. wide at the middle, and raised strips on the bottom of the shoe and on the top of the base plate mesh with this groove. The object of this device is to keep the rollers in line.

The ties are not notched down over the girders, as is often done, but are leveled up (over the varying number of flange cover plates) by blocking under the ties, the blocks being tapered to give the proper superelevation to the outer rail. The blocks are bolted to the ties and are notched over the flanges of the girders, but they are not bolted or otherwise fastened to the girders.

TRANSPORTATION AND ERECTION.—The detail construction of the bridge, though interesting because of the size of the girders, is yet second in interest to the matter of transporting it to the site and putting it in place. Shipping girders of this size in a single piece, and erecting them in place of an existing bridge without appreciable interruption to traffic, is a rare achievement. This work was done by the railroad company itself, to whom the finished parts were delivered at the works of the American Bridge Co., at Elmira, N. Y.

The manner of loading one of the main girders on four flat cars is shown in the photographic view, Fig. 4. The load is carried by the end cars only, but low blocking was provided on the intermediate cars on which the girders would rest in case of any failure in the end supports. The girder was braced in its upright position by diagonal struts on the end supports; these struts were framed and tie-bolted to the transverse timbers underneath to form a cradle. The weight of the cradle and its load was carried by longitudinal timbers which distributed the load.

Moving this shipment over the railway from Elmira, N. Y., to the site was a very delicate job. It was run "special," and over some parts of the line the speed of the train had to be kept down to 4 miles per hour, as the swaying set up prohibited any greater speed. Thanks to careful handling, however, the girders reached their destination in safety.

The bridge was erected complete on land, alongside the track near one end of the crossing, and from here was launched over the stream on timber falsework to a position alongside the old bridge. The latter having been placed on blocking, temporary supports for it were built on the side opposite the new structure, and the masonry abutments were remodeled to suit the new bridge. When everything was ready the old bridge was slid laterally out of the way and the new one

FIG. 4. 133-FT. GIRDER LOADED FOR SHIPMENT.

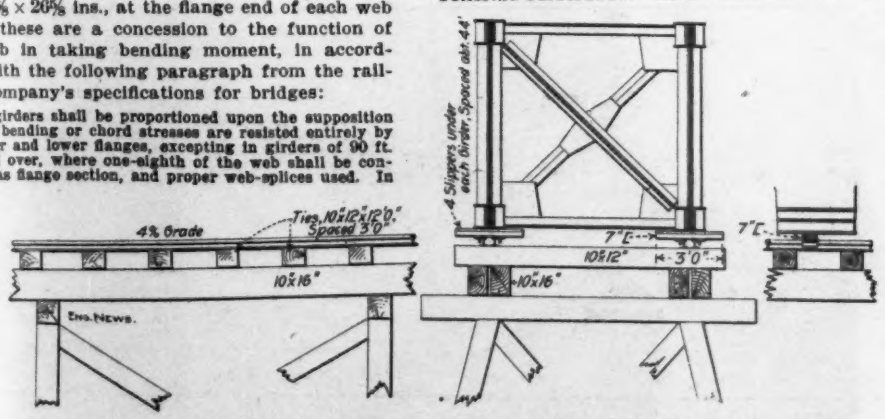


FIG. 5. ARRANGEMENT FOR LAUNCHING BRIDGE.

slid into its place. The moving of each bridge occupied about 10 minutes, and the track was closed a total time of only one hour.

More in detail, the course of the erection work was as follows: A cribwork support for the girders was built alongside the track on shore, near one end of the bridge, to the height of a flat car. The cars bearing one girder were pulled up alongside this cribbing; timbers capped with rails were laid across, and the girder with its cradles was slid over on them to its temporary support. The rails were well lubricated with common machine oil for this purpose, and a pile-driver engine pulling on a line reeved through multiple blocks was used to slide the girder across. The work was complicated by the fact that it was done on a curve in the track, where the outer rail had a superelevation of about 5 ins.

After the second girder was similarly unloaded and set up on the cribbing the laterals and sway frames were put in place between the girders, and riveted up, and ties and rails laid on the top, so that the bridge was complete but not in place. A trestle was built out across the river, alongside the old bridge, and in line with the new structure on the cribbing, on a 4% down grade. Two lines of rails 9 ft. apart, each consisting of two rails side by side, were laid on this trestle to form a runway for the new bridge. Under each girder of the latter were placed four skates, one at each end and two intermediate, which were to slide on the rails. These skates were made of a 7-in. channel, as sketched in Fig. 5. By using machine oil to lubricate the rails, the structure was hauled down the runway, ready to be slid laterally across into place. Meanwhile the old bridge and adjoining track had been raised to the new track grade,

3 ft. higher than the old level; the old abutments were changed to seat the new bridge, and a trestle support had been built on which the old bridge could be slid laterally out of place.

To move the new bridge into place, four lines of rails were laid on each abutment, transversely to the axis of the bridge, at such a level as to bring the bridge into its proper position; the girders rested directly on these rails and the bridge was slid over by a hoisting engine working through a six-part line (1⅝-in. rope) as before, the rails being well oiled. Endwise motion of the bridge was prevented by a rail bolted horizontally against the face of each abutment. The new bridge was opened for traffic as soon as it was in proper alinement; to complete it the only work remaining was to remove the slide rails, place the shoes, and lower the bridges to a bearing on them, which was done while the trains were running.

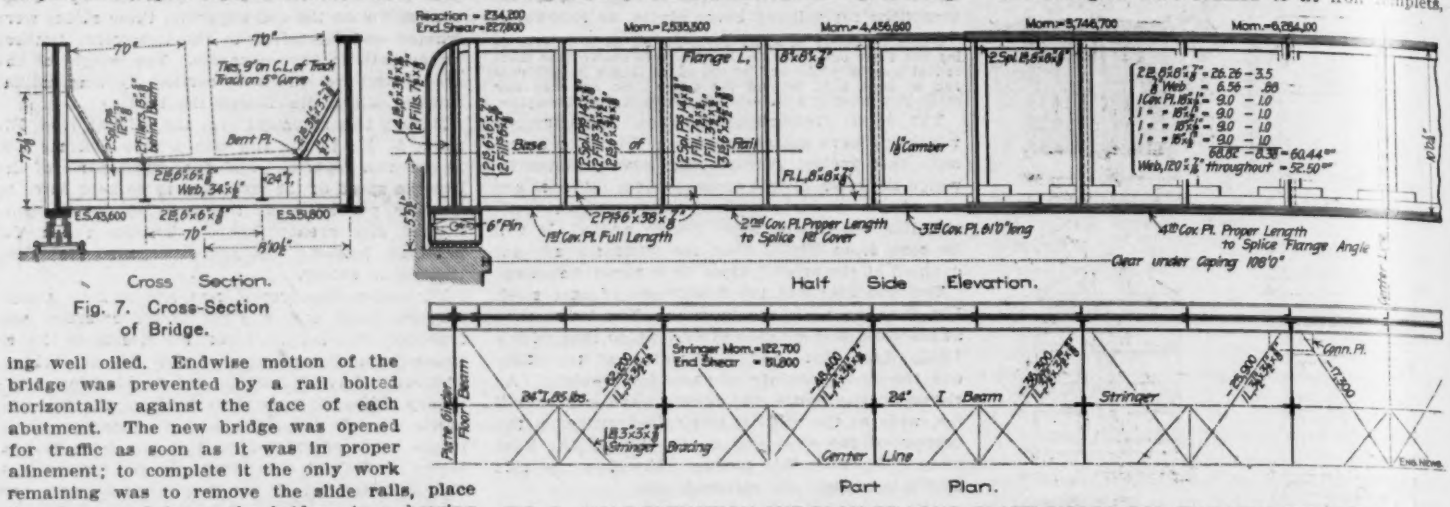


FIG. 6. HALF-ELEVATION AND PLAN OF LONG PLATE-GIRDER FOR THE YELLOW CREEK BRIDGE OF THE ERIE, ALLIANCE & WHEELING RY.
King Bridge Co., Cleveland, O., Builders.

The bridge was designed by the engineering department of the Erie R. R., Mr. M. R. Strong, Bridge Engineer, under the then chief engineer, Mr. C. W. Buchholz. It was built at the Elmira (N. Y.) plant of the American Bridge Co.; the work of erection was done by the railroad itself, through the regular bridge force of the Mahoning division.

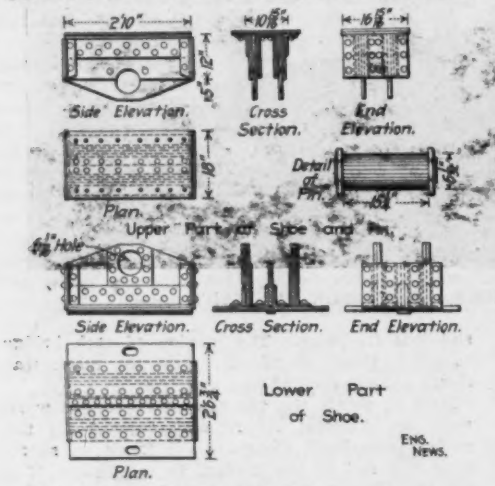


Fig. 8. End Shoe.

The specifications under which the bridge was designed are the regular Erie R. R. specifications (1900). The principal departure from them is in the spacing of the girders, which are 9 ft. apart instead of "a maximum of 7 ft." for deck plate girders, according to the specifications. For this reason the ties, which rest directly on the top flange, are extra heavy, being 10 × 12 ins. in section, set on edge, by 12 ft. long.

The material of the bridge is soft steel throughout, except in the end shoes. A rather unusual feature in the specifications for workmanship is the clause, "No steel more than five-eighths of an inch thick shall be punched and used for tension members. It shall be drilled instead, or punched and reamed." In accordance with this, the bottom flange angles and the bottom cover plates were sub-punched (¾-in.) and reamed (15-16-in., for ⅞-in. rivets), while the top flange, which has the same make-up, is punched 15-16-in.

Though this bridge is the longest plate-girder span on the Erie R. R., it is but very little longer than an earlier one, built in 1890. This latter, Bridge No. 7, of the Bradford Division of the Erie R. R., is a single-track plate-girder span, 128 ft. 3 ins. extreme length, girders 9 ft. 6 ins. deep out to out. Each of the girders of this bridge weighed 98,000 lbs. Like the Yankee Run bridge, this work was fabricated at Elmira and erected by the railway company.

In addition to these, the Erie R. R. has a great number of plate-girder bridges of spans from 100 to 110 ft., whose construction is due largely to the general preference entertained by the engineering department of that railway for plate girders in preference to either through trusses or lattice girders.

Bridge over Yellow Creek; Lake Erie, Alliance & Wheeling Railway.

A single-track through bridge over Yellow Creek, on the line of the Lake Erie, Alliance & Wheeling Ry., is a plate-girder structure 114 ft. 3¾ ins. over all. The girders, 10 ft. ¼-in. deep over flange angles, are spaced 17 ft. 9 ins. apart on centers, and a regular floor beam and stringer construction is used. A notable feature of this bridge is that it lies on a 5° curve. The accompanying drawing, Fig. 6, shows elevation and plan of the bridge; the cross-section is given in Fig. 7.

Each girder has a 7-16-in. web plate, 8 × 8 × ⅞-in. flange angles, and four cover plates 20 × ½-in. The web is stiffened at intervals of about 6 ft. by a single angle on each side of the web. At alternate panel points are plate-girder floor beams, and here the inside stiffeners consist of a pair of angles. The stringers are 24-in. × 85-lb. I-beams.

The end shoes are of riveted steel construction. In order to distribute the weight upon them, a 6-in. pin bearing is used in each shoe, as shown in detail in Fig. 8. Under each shoe at the expansion end of the bridge is a nest of eight 3-in. rollers, 24 ins. long, the middle portion of each roller being turned down to a 2½-in. diameter for a width of 2¼ ins.

The bridge was designed under the specifications of the railway company, and was erected by the contractor, ready for the wood flooring. The material is of soft open-hearth steel except the rollers and hand-driven rivets, which are of iron. This applies to both shop and field rivets driven by hand. The bottom flange angles were reamed, and the field connections in floor beams and stringers were reamed to fit iron templets, thus ensuring that the work would go together properly in erection. The rivets are ⅞-in. diameter, in 15-16-in. holes. Rivet holes requiring reaming were punched ⅛-in. less than the diameter of the rivet and then reamed to 1-16-in. larger than that diameter. The sharp edges of reamed holes and the fins around punched holes were all removed before riveting, and no sharp or unfilleted angles or corners were allowed on any piece of metal. The painting consisted of one shop coat of Sherwin, Williams & Co.'s bridge paint, and another coat of the same after erection. Contact surfaces were painted before the pieces were riveted together, and two shop coats of paint were given to the bottom of bed plates, bearing plates, and other parts not accessible after erection.

Each girder was riveted up complete in the shop, and weighed about 84,640 lbs. when ready for transportation. It was loaded on three steel flat cars, as shown in Fig. 9. Near the inner end of each outer car was placed a bolster on which the girder rested. Timber braces rested on the ends of the bolsters, and had their upper ends fitted against the top chord, being secured by iron straps passing over the top of the girder. Iron side straps also connected the bolster and brace. To prevent lateral movement of the braces, angle iron lugs were riveted to the girder web and to these were attached iron bars running to the braces.

FIG. 9. 114-FT. PLATE-GIRDER READY FOR TRANSPORTATION.